

APPENDIX A

SMALL-BOAT HARBOR MODEL TEST INVENTORY

Section AI. Physical Model Investigations Conducted for
Various Small-Boat Harbor Sites (Classifications)

A-1. General. This part of Appendix A lists small-boat harbors for which physical model investigations were conducted at WES. These sites are grouped into the various harbor classifications (see paragraph 3-23 in main text) and further divided by the nature of the problems studied.

A-2. Open Coast Harbors Built Seaward/Lakeward from the Shoreline and Protected by Breakwaters. Subparagraphs a-e below show the nature of specific problems for which model investigations have been conducted for this class harbor site. Under each of these subparagraphs, a list of specific harbor sites studied is shown.

a. Wave Action Studies (Short-Period Wave Protection).

- (1) Oceanside Harbor, California (Curren and Chatham 1980)*
- (2) Port Washington Harbor, Wisconsin (Bottin 1976, 1977) (Fortson et al. 1951)
- (3) Jubail Harbor, Saudi Arabia (Giles and Chatham 1976)
- (4) Waianae Harbor, Hawaii (Bottin, Chatham, and Carver 1976)
- (5) Agana Harbor, Guam (Chatham 1975)
- (6) Port Orford, Oregon (Giles and Chatham 1974)
- (7) Tau Harbor, American Samoa (Crosby 1974)
- (8) Crescent City Harbor, California (Senter 1971) (Senter and Brasfeild 1968)
- (9) Port San Luis, California (Chatham and Brasfeild 1969)
- (10) Monterey Harbor, California (Chatham 1968) (Fortson et al. 1949)
- (11) Kawaihae Harbor, Hawaii (Brasfeild and Chatham 1967)
- (12) Magic Island Complex, Hawaii (Brasfeild and Chatham 1967)

* See Bibliography (Appendix B).

25 Sep 84

- (13) Santa Barbara Harbor, California (Brasfeild and Ball 1967)
- (14) Dana Point Harbor, California (Wilson 1966)
- (15) Half-Moon Bay Harbor, California (Wilson 1965)
- (16) Conneaut Harbor, Ohio (Hudson and Wilson 1963)
- (17) Lorain Harbor, Ohio (Wilson, Hudson, and Housley 1963)
- (18) Barcelona Harbor, New York (Jackson, Hudson, and Housley 1959)
- (19) East Beaver Bay Harbor, Minnesota (Fortson et al. 1949)
- (20) Oswego Harbor, New York (Fortson et al. 1949)
- (21) Anaheim Bay, California (Brown, Hudson, and Jackson 1948)

b. Shoaling Studies (Shoaling Protection).

- (1) Oceanside Harbor, California (Curren and Chatham 1980)
- (2) Waianae Harbor, Hawaii (Bottin, Chatham, and Carver 1976)
- (3) Port Orford, Oregon (Giles and Chatham 1974)

c. Wave-Induced Circulation/Current Studies.

- (1) Port Washington, Wisconsin (Bottin 1977)
- (2) Agana Harbor, Guam (Chatham 1975)
- (3) Tau Harbor, American Samoa (Crosby 1974)
- (4) Kawaihae Harbor, Hawaii (Brasfeild and Chatham 1967)
- (5) Magic Island Complex, Hawaii (Brasfeild and Chatham 1967)
- (6) Monterey Harbor, California (Chatham 1968)
- (7) Lorain Harbor, Ohio (Wilson, Hudson, and Housley 1963)

d. Long-Period Harbor Oscillation Studies.

- (1) Monterey Harbor, California (Chatham 1968) (Fortson et al. 1949)
- (2) Anaheim Bay, California (Brown, Hudson, and Jackson 1948)

25 Sep 84

e. Standing Waves (Short-Period Generated).

- (1) Port Washington Harbor, Wisconsin (Bottin 1976, 1977)

A-3. Harbors Build Inland with an Entrance Through the Shoreline. Subparagraphs a-e below, give the nature of various problems for which model investigations have been conducted for this class harbor site. These subparagraphs are further divided to list the specific harbor sites studied.

a. Wave Action Studies (Short-Period Wave Protection).

- (1) Geneva-on-the-Lake Harbor, Ohio (Bottin 1982)
- (2) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)
- (3) Mission Bay Harbor, California (Curren 1983) (Ball and Brasfeild 1969)
- (4) Kewalo Basin, Hawaii (Giles 1975)
- (5) Ludington Harbor, Michigan (Crosby and Chatham 1975)
- (6) Hamlin Beach, New York (Brasfeild 1973)
- (7) In-Shore Harbor, Site X, South China Sea (Wilson 1966)
- (8) Marina Del Rey, California (Brasfeild 1965)
- (9) Grand Marais Harbor, Minnesota (Fenwick 1944) (Schroeder and Easterly 1941)

b. Shoaling Studies (Shoaling Protection).

- (1) Geneva-on-the-Lake, Ohio (Bottin 1982)
- (2) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)
- (3) Mission Bay Harbor, California (Curren 1982)

c. Wave-Induced Circulation/Current Studies.

- (1) Geneva-on-the-Lake, Ohio (Bottin 1982)
- (2) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)
- (3) Mission Bay Harbor, California (Curren 1982)

25 Sep 84

(4) Kewalo Basin, Hawaii (Giles 1975)

(5) Ludington Harbor, Michigan (Crosby and Chatham 1975)

d. Long-Period Harbor Oscillation Studies.

(1) Mission Bay Harbor, California (Ball and Brasfeild 1969) (Curren 1982)

(2) Port Hueneme, California (Crosby, Durham and Chatham 1975)

e. Seiche Studies.

(1) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)

A-4. Harbors Built Inside a River/Stream Mouth. Subparagraphs a-e below depict the nature of various problems for which model tests have been conducted for this class harbor site. Further division of these subparagraphs lists specific harbor sites studied.

a. Wave Action Studies (Short-Period Wave Protection).

(1) Rogue River, Oregon (Bottin 1982)

(2) Port Ontario Harbor, New York (Bottin 1977)

(3) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

(4) Chagrin River, Ohio (Chatham 1970)

(5) Vermilion Harbor, Ohio (Brasfeild 1970)

(6) New Buffalo Harbor, Michigan (Dai and Wilson 1967)

(7) Noyo Harbor, California (Wilson 1967)

b. Shoaling Studies (Shoaling Protection).

(1) Rogue River, Oregon (Bottin 1982)

(2) Siuslaw River, Oregon (Bottin 1981)

(3) Port Ontario Harbor, New York (Bottin 1977)

(4) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

c. Wave-Induced Circulation/Current Studies.

(1) Rogue River, Oregon (Bottin 1982)

- (2) Port Ontario Harbor, New York (Bottin 1977)
- (3) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)
- (4) Chagrin River, Ohio (Chatham 1970)

d. Riverflow/Flood Control Studies.

- (1) Rogue River, Oregon (Bottin 1982)
- (2) Port Ontario Harbor, New York (Bottin 1977)
- (3) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)
- (4) Chagrin River, Ohio (Chatham 1970)

e. Ice-jamming Studies.

- (1) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

A-5. Entrance/Inlet Studies. Physical model investigations conducted for this class of harbor site deal primarily with navigation at the entrance to the inlet. Subparagraphs a-f below, show the nature of specific problems for which model investigations have been conducted for this class of harbor site. These subparagraphs are further divided to depict specific harbor sites studied.

a. Wave Action Studies (Short-Period Waves in Entrance).

- (1) Oregon Inlet, North Carolina (Seabergh, Hollyfield, and McCoy 1983)
- (2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)
- (3) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978)
- (4) Wells Harbor, Maine (Bottin 1978)
- (5) Little River Inlet, South Carolina (Seabergh and Lane 1977)
- (6) Masonboro Inlet, North Carolina (Seabergh 1976)
- (7) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974)
- (8) Nassau Harbor, Bahamas (Brasfield 1965)

b. Shoaling Studies (Entrance Shoaling Protection).

(1) Oregon Inlet, North Carolina (Seabergh, Hollyfield, and McCoy 1982)

(2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)

(3) Little River Inlet, South Carolina (Seabergh and Lane 1977)

(4) Masonboro Inlet, North Carolina (Seabergh 1976)

(5) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974)

c. Wave-Induced Circulation/Current Studies.

(1) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)

(2) Wells Harbor, Maine (Bottin 1978)

d. Tidal Circulation/Flood and Ebb Currents.

(1) Oregon Inlet, North Carolina (Seabergh, Hollyfield, and McCoy 1982)

(2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)

(3) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978)

(4) Wells Harbor, Maine (Bottin 1978)

(5) Little River Inlet, South Carolina (Seabergh and Lane 1977)

(6) Masonboro Inlet, North Carolina (Seabergh 1976)

(7) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974)

(8) Nassau Harbor, Bahamas (Brasfield 1965)

e. Tidal Elevation Studies (Water-Surface).

(1) Oregon Inlet, North Carolina (Seabergh, Hollyfield, and McCoy 1982)

(2) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978)

(3) Little River Inlet, South Carolina (Seabergh and Lane 1977)

(4) Masonboro Inlet, North Carolina (Seabergh 1976)

(5) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974)

f. Salinity Studies.

- (1) Little River Inlet, South Carolina (Seabergh and Lane 1977)

Section AII. Hydraulic Model Investigations Conducted
for Various Sites (Case Histories)

A-6. General. This section of Appendix A discusses typical small-boat harbors in each harbor classification. Physical model investigations were conducted to determine solutions for various problems for these harbors which are located on the various ocean coasts and/or the Great Lakes. The sites discussed for each harbor classification are as follows:

a. Open coast harbors built seaward/lakeward from the shoreline and protected by breakwaters.

- (1) Dana Point Harbor, California (Wilson 1966)
(2) Port Washington Harbor, Wisconsin (Bottin 1976, 1977)

b. Harbors built inland with an entrance through the shoreline.

- (1) Mission Bay Harbor, California (Curren 1982)
(2) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)

c. Harbors built inside a river/stream mouth.

- (1) Rogue River Harbor, Oregon (Bottin 1982)
(2) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

d. Entrance/inlet studies.

- (1) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)
(2) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978)

A-7. Open Coast Harbors Built Seaward/Lakeward from the Shoreline and Protected by Breakwaters. Numerous small-craft harbors of this type are constructed along the ocean coasts and Great Lakes' shorelines. Dana Point Harbor, California, located on the Pacific Coast, and Port Washington Harbor, Wisconsin, situated on the western shore of Lake Michigan, were selected as representative harbors under this classification and are discussed below.

a. Dana Point Harbor, Dana Point, California (Wilson 1966).

- (1) The Prototype. At the time of the hydraulic model investigation,

25 Sep 84

Dana Point, California, was the proposed site for a small-boat harbor, located in Orange County on the Southern California coast about 40 miles southeast of the Los Angeles-Long Beach harbors (Figure A-1). The proposed harbor site was in a sheltered cove in the lee of the Dana Point promontory. Dana Cove is a very scenic area, and the existing pier and beach attract many sport fishermen, sun bathers, and surfers. The proposed small-boat harbor at Dana Point was one of a chain of small-craft harbors to be constructed along the California coast under the program of Federal and local government cosponsorship of small-craft harbors and harbors of refuge. After ultimate development, the enclosed harbor would enclose an area of about 210 acres. Within this area, facilities would accommodate the berthing and servicing of about 2,150 small boats.

(2) The Problem. Dana Cove is protected from northwest, north, and northeast windstorms by comparatively high bluffs along the shoreline. The Santa Catalina and San Clemente Islands also provide some protection from

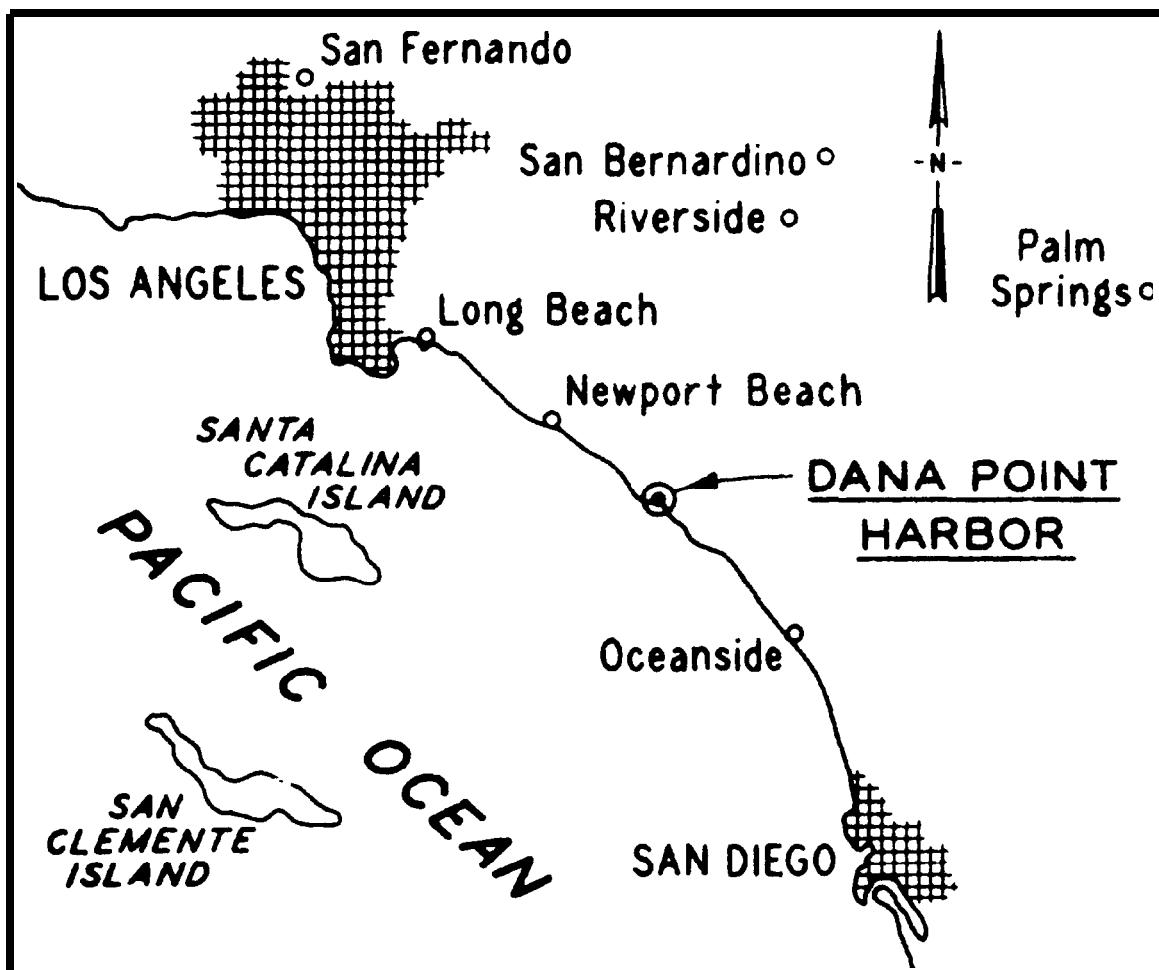


Figure A-1. Project location, Dana Point, California.

25 Sep 84

storm waves from the west to southwest directions. The cove, however, is exposed to storm waves from directions ranging counterclockwise between southwest and south-southeast and to ocean swells from the south. Waves breaking on the Dana Point shoreline normally range from about two to four feet. However, waves ranging from about six to ten feet are not uncommon and may occur during any season of the year. Over a 65-year period of record, waves reaching Dana Cove attained a significant height of 16 feet twice and a significant height of 26 feet once.

(3) The Model and Test Conditions. A physical model investigation was conducted to evaluate the adequacy of design of the proposed plan of harbor development to ensure that optimum navigability, maneuverability, and wave protection were provided for pleasure craft during storm-wave attack, all at minimum cost. The Dana Point Harbor model (Figure A-2) was constructed to an undistorted linear scale of 1:100, model to prototype. Model test waves with periods ranging from 5 to 18 seconds and heights ranging from 7 to 16 feet are shown in Table A-1. A still-water level of +6.0 feet mllw [mean higher high water (+5.3 feet) plus a wind tide of 0.7 foot] also was used during model testing.

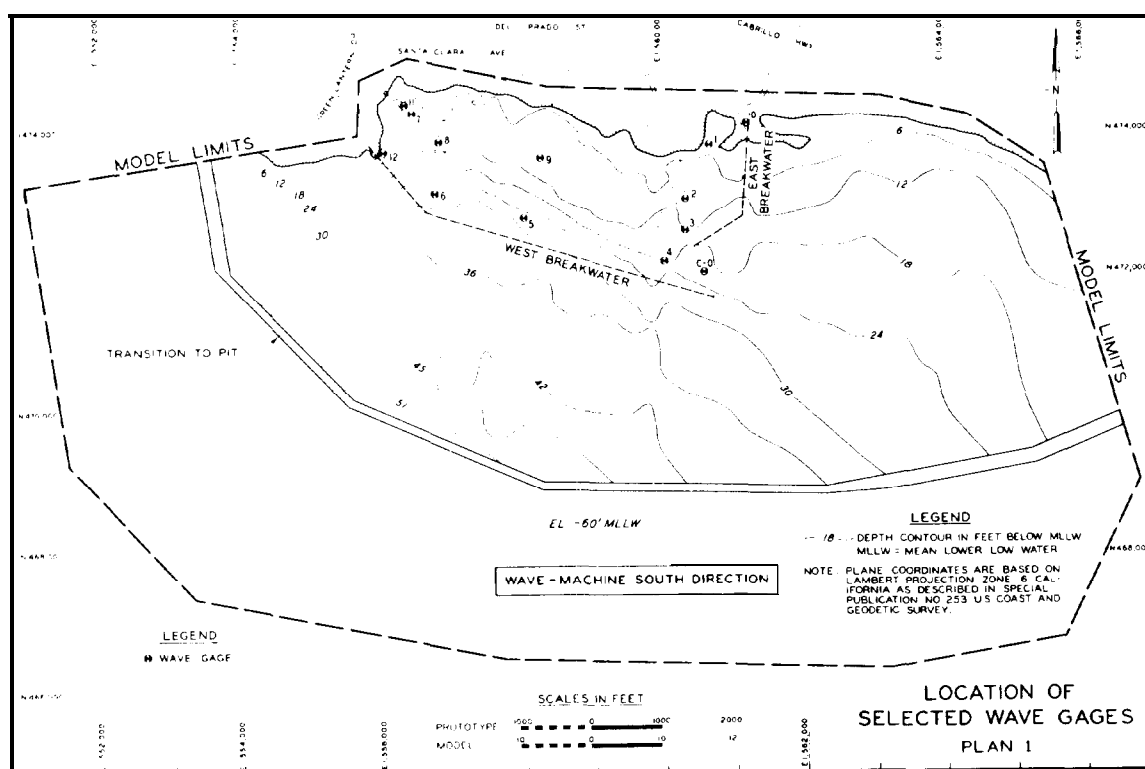


Figure A-2. Model layout, Dana Point Harbor.

TABLE A-1

Test Waves Used in the Dana Point Harbor Model
(USAED, LA 1961) (Marine Advisors 1960, 1961)

<u>Deepwater Direction</u>	<u>Selected Test Waves</u>	
	<u>Period (sec)</u>	<u>Height (ft)*</u>
N 80° W	13	9
West	9 18	7, 11 7
S 70° w	10	7, 11
S 65° W	7	9
S 60° W	15	7
S 45° W	9 12	9 6, 14
S 25° W	12 14 18	7, 14 16 7
S 5° W	7	11
South	11	7, 14
S 10° E	18	7
S 12° E	9	7, 13
S 22 1/2° E	5 11	7 7, 14
S 30° E	7	10
S 40° E	9	7, 11

*Wave heights shown are shallow-water values (adjusted as a result of refraction-shoaling analysis).

(4) Tests and Results.

(a) Existing Conditions. Prior to tests of the various improvement plans, wave height tests were conducted to determine the general wave conditions in the area proposed for the harbor. Results of these tests indicated very rough and turbulent conditions in the area of the proposed harbor. Wave heights adjacent to an existing pier well within the proposed harbor were almost seven feet.

(b) Improvement Plans. Wave height tests were conducted for 13 variations in the design elements of the basic improvement plan. Variations consisted of changes in the breakwater cross-sections and alignments, installation of vertical piers in the harbor, and the omission of the west-basin berthing development and mole section. Initially, tests were conducted for only the first step in the development of the proposed harbor and consisted of an aggregate length of breakwater structure of 7,750 feet (Figure A-3). Observations of

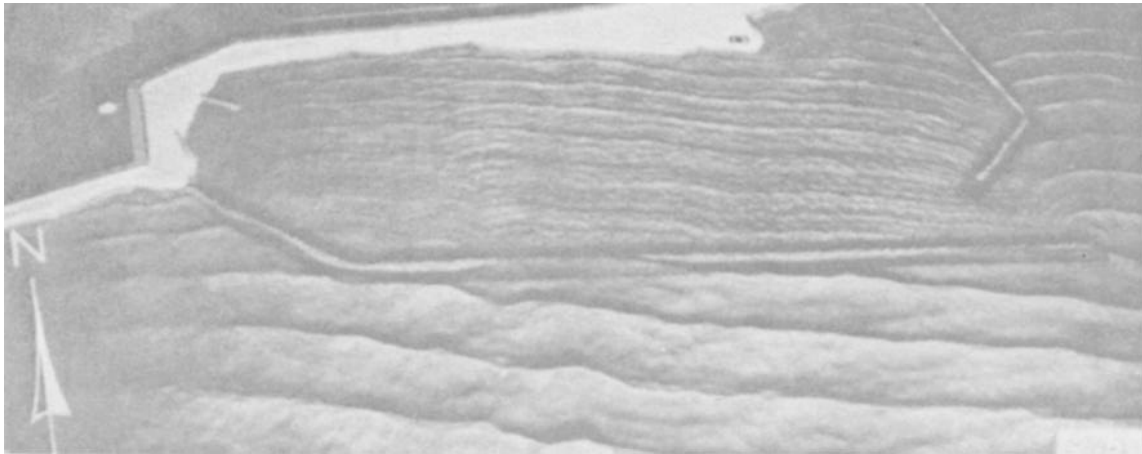


Figure A-3. Wave patterns for the initial step of development for the proposed harbor, Dana Point model.

these tests revealed significant overtopping of the structures and test results indicated the required four-foot wave height criteria in the approach channel of the proposed harbor was exceeded. Next, the proposed inner harbor complex was installed in the model. This consisted of east and west berthing areas, enclosed by mole sections, and connected by a 200-foot-wide, 10-foot-deep navigation channel. A 350-foot-wide fairway channel, a ramp area, refuge area, and recreational facilities were also included. Based on test results, modifications were made to the breakwater crest elevations, lengths, and alignments until a plan was developed that provided adequate wave protection in the fairway and approach channels, ramp area, and mooring areas (Figure A-4). Tests were conducted in the model to determine the effect of a vertical face pier installed in the western sector of the harbor. This pier would be used

25 Sep 84



Figure A-4. Wave patterns for the recommended improvement plan, Dana Point model.

as a boat repair facility should future need arise. As a result of this modification it was determined that wave action would not significantly increase in this section of the harbor. The west-basin berthing development and mole section were removed to determine the amount of protection that would be provided against storm waves from southwest should only the east basin berthing area be constructed in the prototype. Test results indicated that wave protection in the harbor would be adequate for this harbor configuration. Subsequent to the model investigation, the harbor was constructed in the prototype at Dana Point, California (Figure A-5) in accordance with recommendations provided, and has functioned quite well, as evidenced by its very heavy usage.

b. Port Washington Harbor, Wisconsin (Bottin 1976, 1977).

(1) The Prototype. Port Washington, Wisconsin, is located on the west shore of Lake Michigan, about 29 miles north of Milwaukee and 27 miles south of Sheboygan (Figure A-6). The city, which had a population of 8,700 in 1970 (USAED-C, 1974) is a trading center and the seat of Ozaukee County. The downtown portions of the business and manufacturing sections have been developed around the harbor. The present harbor is entirely artificial and located at the outlet of a small stream known as Sauk Creek. The harbor area comprises

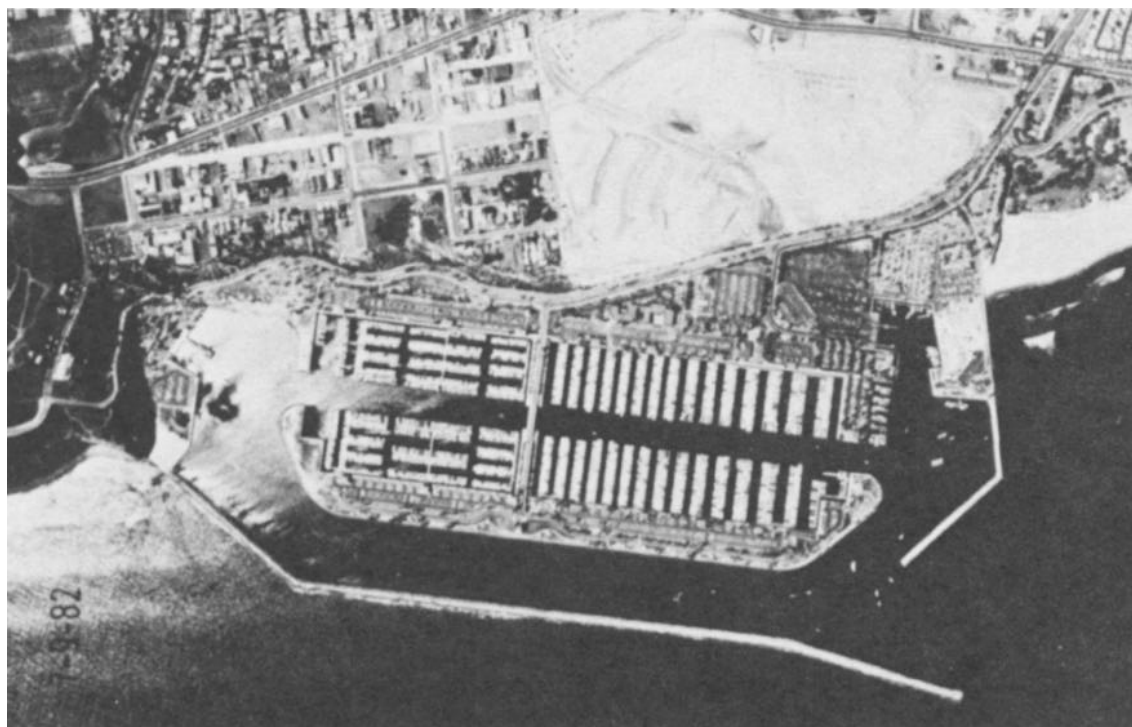


Figure A-5. Aerial photo of Dana Point Harbor, California.

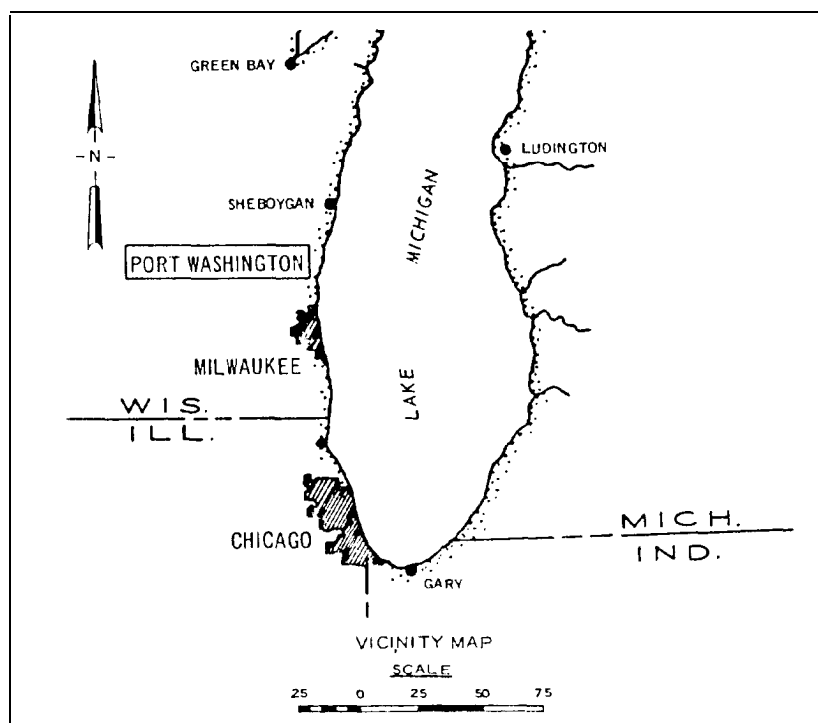


Figure A-6. Project location, Port Washington Harbor, Wisconsin.

approximately 60 acres and is enclosed by a 3500-foot-long breakwater system (Figure A-7). The outer harbor is maintained at a project depth of 21 feet and the inner harbor or slip area, is maintained at a project depth of 18 feet.

(2) The Problem. Port Washington Harbor is exposed to waves generated by storms from northeast clockwise to south-southeast. Waves due to storms from these directions have caused considerable damage to harbor facilities and recreational boats and created difficulties for ships and recreational craft navigating the harbor entrance. Violent wave action, caused by waves reflected from vertical steel sheet-pile bulkheads, has resulted in wave heights up to 12 feet in the slip areas of the inner harbor. Anchorage in the outer basin is not safe for small boats because of the lack of adequate wave protection. These conditions made the harbor unsafe as a harbor-of-refuge for small boats, resulting in no adequate small-boat refuge between Milwaukee and Sheboygan, a distance of 56 miles. In addition, there was a lack of adequately protected permanent mooring and docking facilities to accomodate the great demand for such facilities in the Port Washington area.

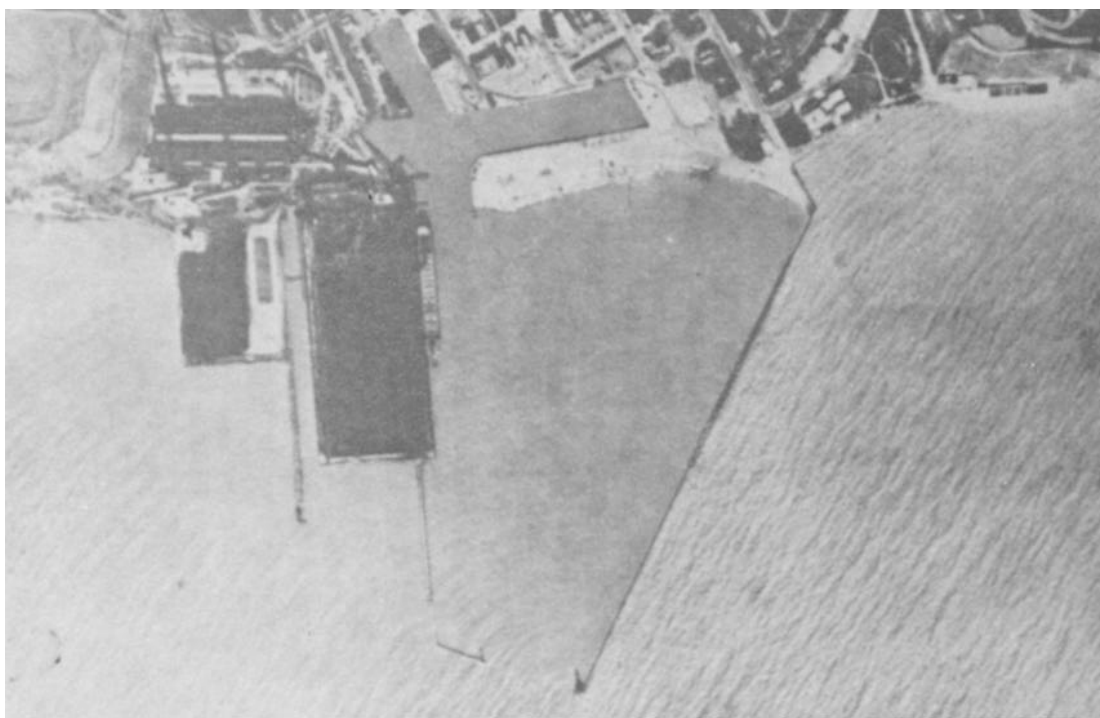


Figure A-7. Aerial photograph of Port Washington Harbor prior to improvements.

(3) The Model and Test Conditions. A physical model investigation was conducted to evaluate the effects of proposed harbor improvements with respect to wave and current conditions in the harbor while minimizing construction costs. The primary improvement was the construction of a small-boat harbor in the northern portion of the existing outer harbor. The Port Washington Harbor model (Figure A-8) was constructed to an undistorted linear scale of 1:75, model to prototype. Model test waves with periods ranging from 5.5 to 10.4 seconds and heights ranging from 3.4 to 14.7 feet are shown in Table A-2. A still-water level of +3.9 feet lwd (low water datum) was selected for use during model testing. This value was obtained from lake stage frequency curves for Milwaukee and Sturgeon Bay, Wisconsin, for a 10-year recurrence interval during the boating season (May-October). A water circulating system was used in the model to reproduce to scale the intake and discharge of cooling water from the Wisconsin Electric Power Company plant. Igloo wave absorber units were installed in the model to determine wave conditions in the inner harbor. These units were tested also as an alternative to rubble-mound breakwaters and

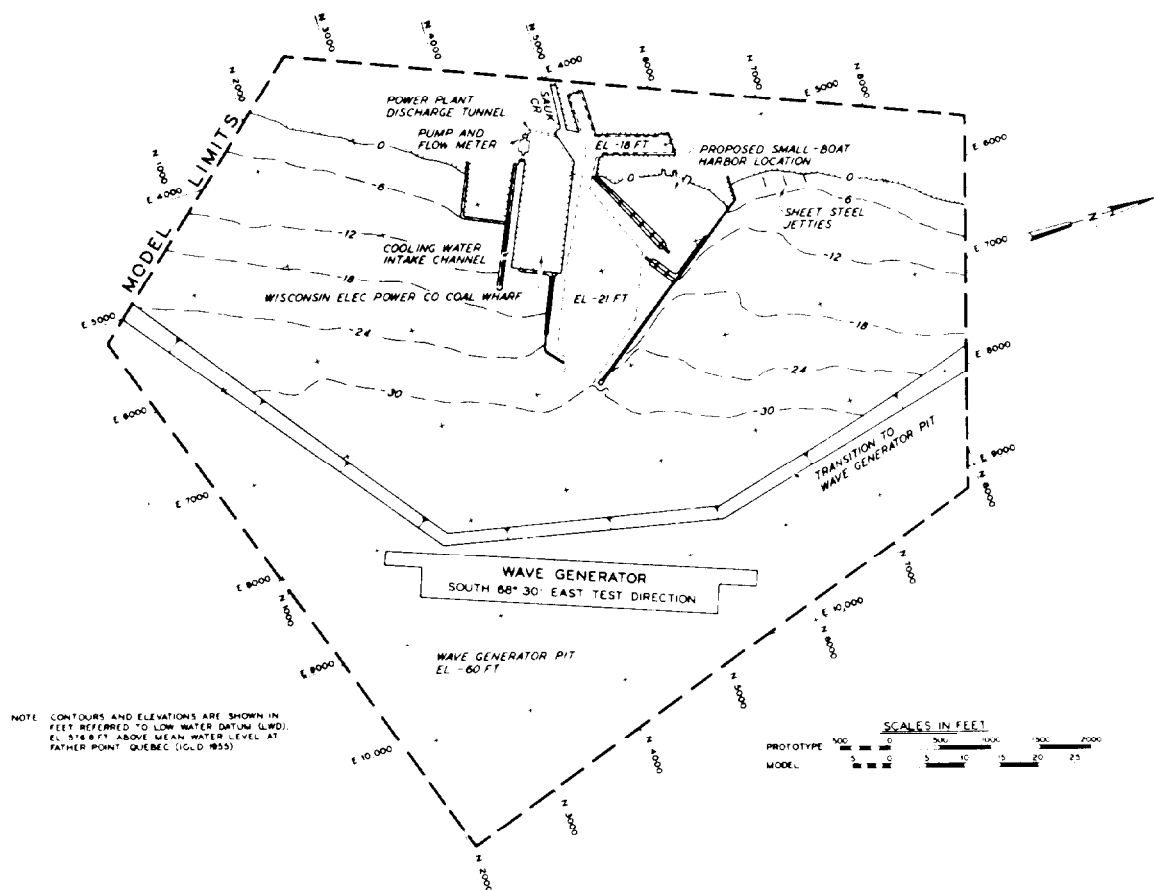


Figure A-8. Model layout, Port Washington Harbor.

TABLE A-2

Test Waves Used in the Port Washington Harbor Model
(Resio and Vincent, Nov 1976)

<u>Deepwater Direction</u>	<u>Shallow-water* Direction</u>	<u>Wave Period (sec)</u>	<u>Deepwater Wave Height (ft)</u>	<u>Shallow-Water" Wave Height (ft)</u>	<u>Recurrence Interval (years)</u>
NE & ENE	N76°20'E	6.0	4.7	4.3	5.1
		7.7	5.0	4.2	6.9
		7.7	9.2	7.7	20
		10.4**	17.1**	14.7**	20
East	S85°50'E	5.5	4.0	3.8	0.33
		7.3	6.0	5.3	6.6
		7.3	10.8	9.6	20
		8.2**	14.8**	12.7**	20
ESE	S68°30'E	5.5	4.0	3.8	0.33
		7.3	6.0	5.5	6.6
		7.3	10.8	9.9	20
		8.2**	14.8**	13.5**	20
SE	S50°45'E	5.5	4.0	3.8	0.33
		7.3	6.0	5.5	6.6
		7.3	10.8	9.9	20
		8.2**	14.8**	13.6**	20
SSE	S37°10'E	6.0	4.4	3.7	1.6
		8.3	4.0	3.4	5.3
		8.3	8.0	6.9	5.4
		8.3	12.1	10.4	20
		9.4**	15.7**	13.8**	20

* Shallow-water values result from refraction-shoaling analysis.

** Wave characteristics for the entire year. All others for spring and summer only.

absorbers in the proposed small-boat harbor. A general view of the model is shown in Figure A-9.

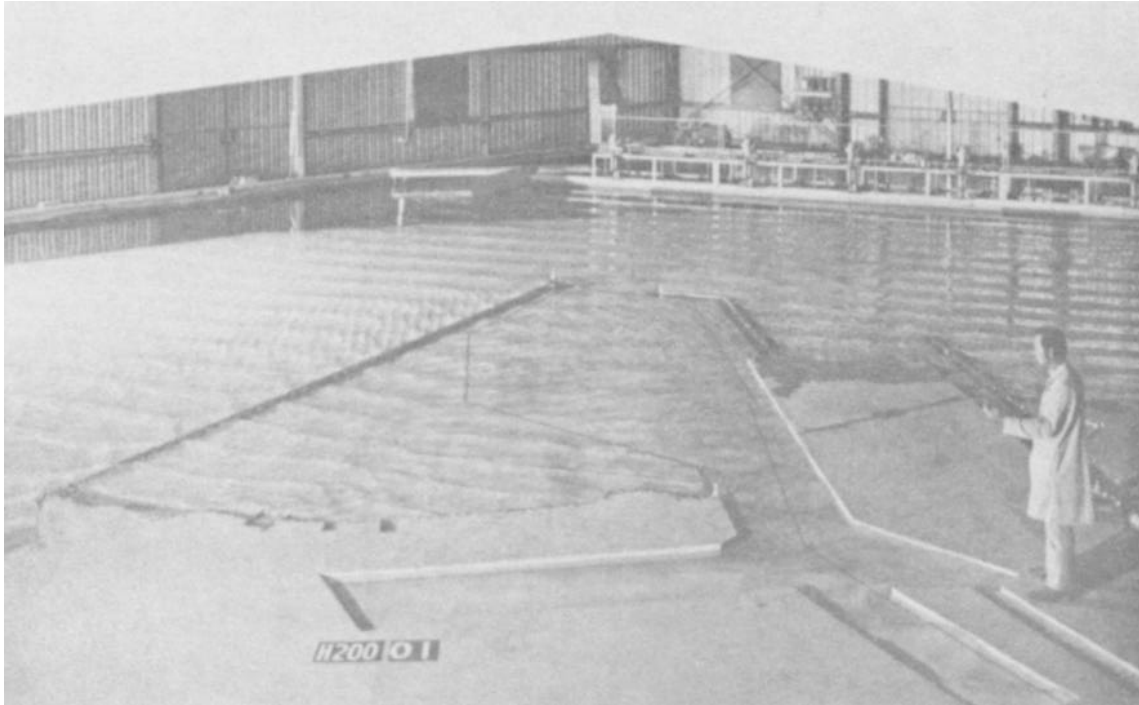


Figure A-9. General view of model, Port Washington Harbor.

(4) Tests and Results.

(a) Existing Conditions. Prior to tests of various improvement plans, comprehensive tests were conducted to determine wave and current conditions in the existing harbor. Test results indicated rough and turbulent conditions in the existing harbor while under storm wave attack. Wave heights in the mooring area of the proposed small-boat harbor exceeded 8 feet in some instances. Also, maximum wave heights in excess of 20 feet were recorded at the coal wharf; and wave heights up to 15 feet were obtained in the inner slip areas of the existing harbor.

(b) Improvement Plans. Wave height tests were conducted for 32 variations of the originally proposed harbor design. Variations included modifications to that portion of the existing north breakwater adjacent to the proposed small-boat harbor and to the proposed east and west breakwaters. Modifications to the north breakwater included raising the crest elevation, installing rubble-mound absorber plans, using the existing breakwater as a core for a rubble-mound breakwater, and installing a concrete parapet wall on the existing breakwater. Modifications to the proposed east and west

25 Sep 84

breakwaters consisted of changes in the crest elevation, alignments, breakwater heads, cross sections of the structures, and the lengths. In addition, wave height tests were conducted for nine test plans which entailed the installation of Igloo absorber units at various locations in the slip areas and as alternatives to the originally proposed rubble-mound breakwaters. (These tests were conducted for Nippon Tetrapod Co., Ltd., after completion of the Corps sponsored investigation.) Wave heights obtained for the originally proposed plan of improvement exceeded the established criteria (a maximum of 2.0 feet in the turning basin and 1.0 foot in the mooring area) for test waves from all test directions. Observations revealed this was due to overtopping of the existing north breakwater (adjacent the harbor) and overtopping of and transmission through the proposed east and west breakwaters. After many alternatives were tested, it was determined that the installation of the concrete parapet wall on the existing north breakwater (adjacent to the harbor) and the modification of the new east and west breakwaters by raising and/or sealing (installing an impervious center) the structures were optimum with respect to economics and wave protection. Also, the removal of 185 feet from the shore end of the west breakwater increased circulation (which should aid in harbor flushing) without increasing wave heights in the proposed harbor. The recommended improvement plan is shown in Figures A-10 and A-11. This plan resulted in wave heights at



Figure A-10. Wave patterns, current patterns, and current magnitudes (fps) for the recommended improvement plan, Port Washington model.

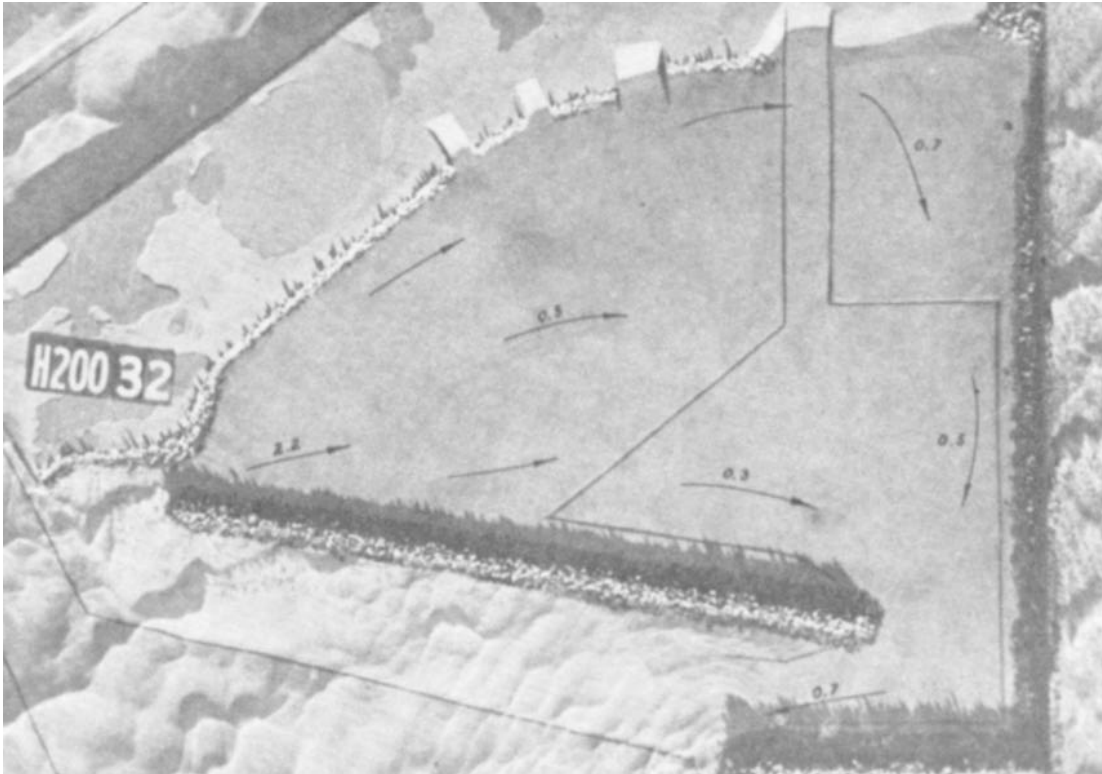


Figure A-11. Closer view of recommended improvement plan, Port Washington Harbor model.

the coal wharf comparable to those obtained for existing conditions; and wave heights along the center line of the slips were, in general, reduced for the recommended plan. Test results with the Igloo wave absorber units placed in and around the slip areas of the existing harbor revealed significantly reduced wave heights in those slips. However, using these units as alternatives to the east and west breakwater revealed that they were not stable in that they required some sort of backing. Construction of the recommended improvement plan in the prototype was completed in 1980 (Figure A-12), and subsequent storms have tested its adequacy. According to the Ozaukee Press (1980) the new small-boat harbor passed with flying colors. The newspaper termed the new harbor as "an oasis of calm assaulted ineffectually by rough seas on three sides." The older portions of the harbor were roiled by waves driven by strong onshore winds, the article said.

A-8. Harbors Built Inland with an Entrance Through the Shoreline. Small-boat harbors of this type are constructed along the ocean coasts and the Great Lakes' shorelines. Mission Bay Harbor, California, located on the Pacific Coast, and Little Lake Harbor, Michigan, situated on Lake Superior, are typical

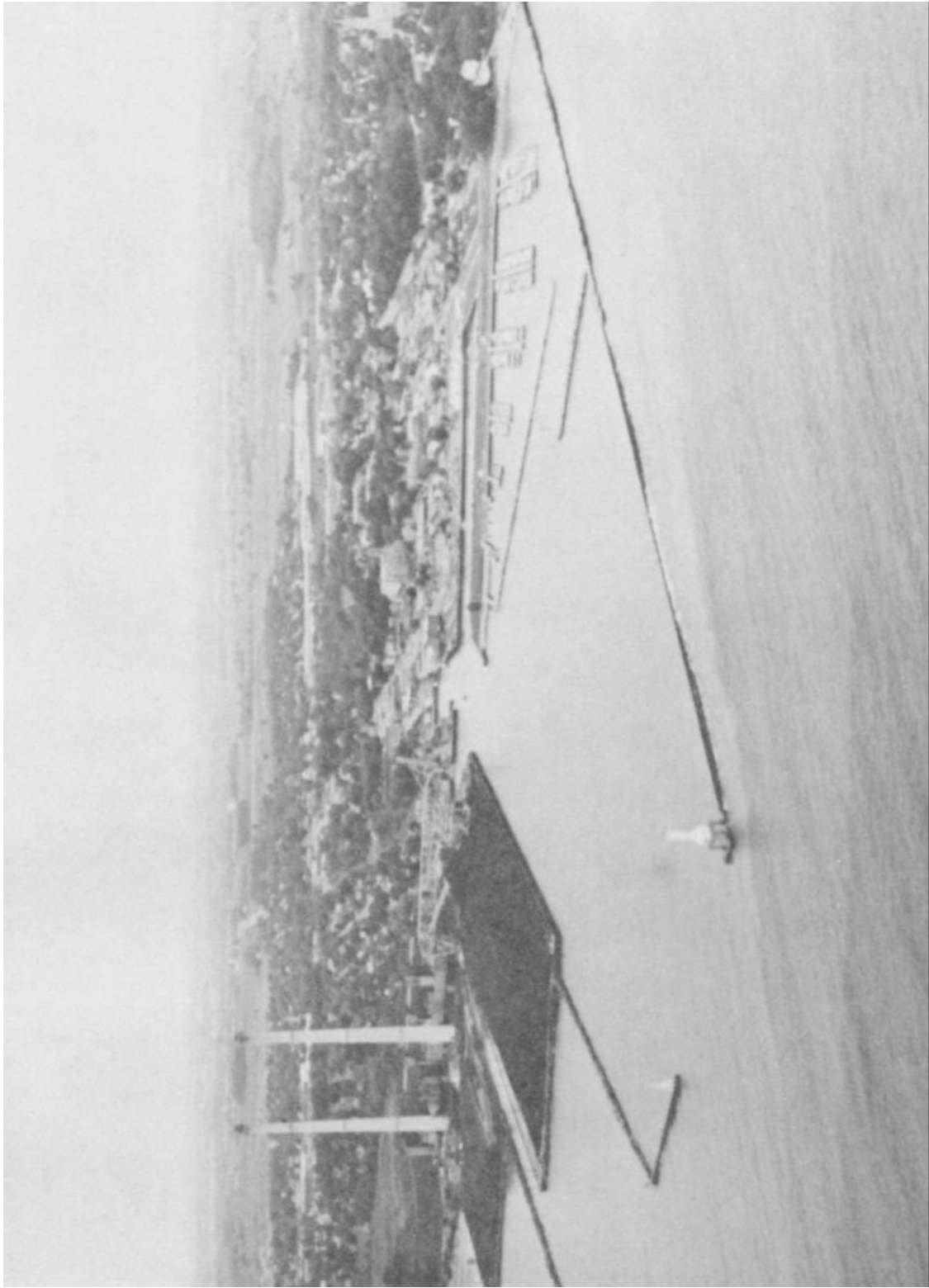


Figure A-12. Aerial photograph of Port Washington Harbor after improvements

examples of small-craft harbors under this classification and are discussed below.

a. Mission Bay Harbor, California (Curren 1982).

(1) The Prototype. Mission Bay Harbor is located on the coast of southern California about 10 miles north of the entrance to San Diego Bay (Figure A-13). The coastline is characterized by gently sloping underwater contours

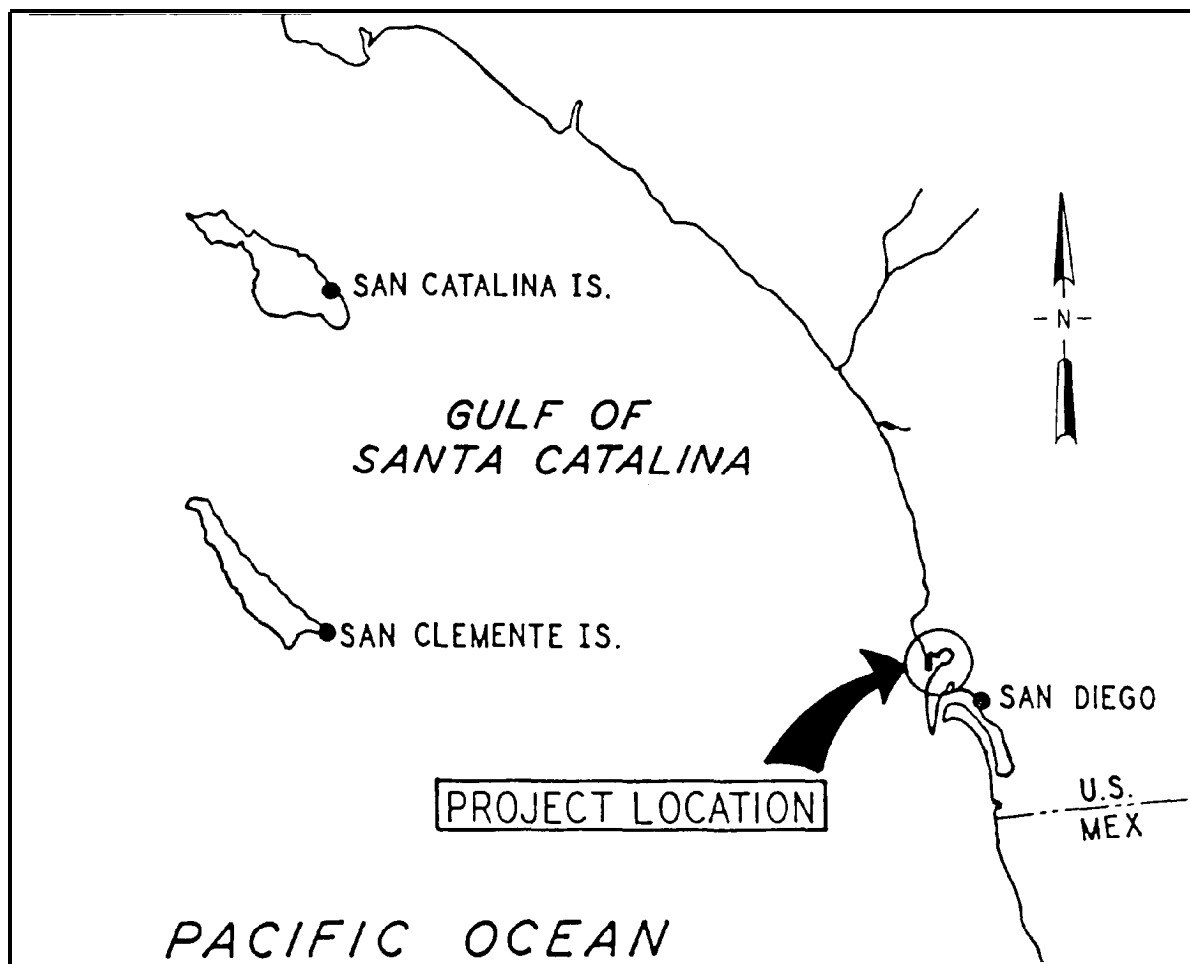


Figure A-13. Project location, Mission Bay Harbor, California.

and sandy beaches. The harbor entrance is protected by two jetties (designated north jetty and middle jetty) extending approximately 3,800 and 4,600 feet into the bay, respectively. Adjacent to the middle jetty is the San Diego River Flood Control Channel which is bounded on the south by the south jetty (Figure A-14). The bay has an effective area of 2,000 acres of navigable water and an

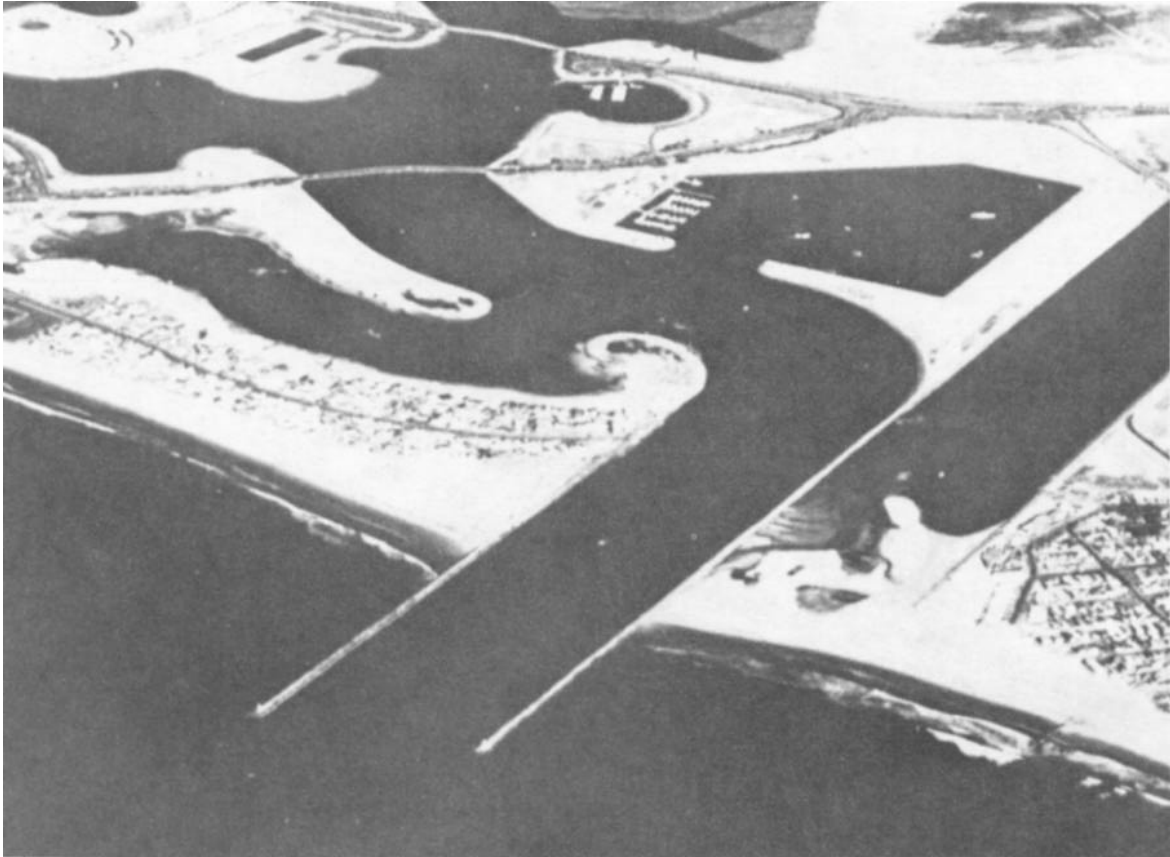


Figure A-14. Entrance to Mission Bay Harbor and view of San Diego River Flood Control Channel.

equal area of land. It is essentially a shallow-draft harbor consisting entirely of recreational and sport-fishing craft.

(2) The Problem. There are basically three problems or potential problems being experienced. They are as follows:

(a) Short-Period Waves. Short-period (less than 20 seconds) waves are breaking in the entrance channel creating hazardous navigation and excessive wave energy in Quivira and Mariners Basins.

(b) Long-Period Waves. Long-period (30-130 seconds) waves are creating oscillations in Quivira and Mariners Basins which excite the floating dock system causing damage to boats and docks, and revetments.

(c) River Shoaling. The mouth of the San Diego River Flood Control Channel is usually blocked by a sand plug (Figure A-14). Normal river flows are

too small to keep a channel open. However, the presence of the plug may be potentially dangerous during a flood. It is uncertain whether the sandplug will wash out rapidly during a flood, or whether the plug will cause a backup of water, resulting in upstream flooding.

(3) The Model and Test Conditions. A physical model investigation was conducted to evaluate the effect of an offshore breakwater on both long and short period wave energy entering the harbor and to evaluate various plans for flood control. The Mission Bay Harbor Model (Figure A-15) was constructed to an undistorted linear scale of 1:100, model to prototype. Model test waves are shown in Table A-3.

TABLE A-3

Test Waves Used in the Mission Bay Harbor Model
(National Marine Consultants, 1960, Marine Advisors, 1961)

<u>Deepwater Direction</u>	<u>Shallow-Water Direction</u>	<u>Selected Test Wave</u>	
		<u>Period (sec)</u>	<u>Height (ft)</u>
NW(310°)	295°	7	6,9
		9	6,9,13
		11	6,9,15
		13	6,11,17
		15	6,11,17
		17	6,11,15
		19	6
W(270°)	267°	7	6,9
		9	6,7,11
		11	6,7,13
		13	6,7,15
		15	6,7,15
		17	6,13
		19	6
SW(220°)	234°	7	6
		9	6,11
		11	6,11
		13	6,11
		15	6,9
		17	6
		19	6

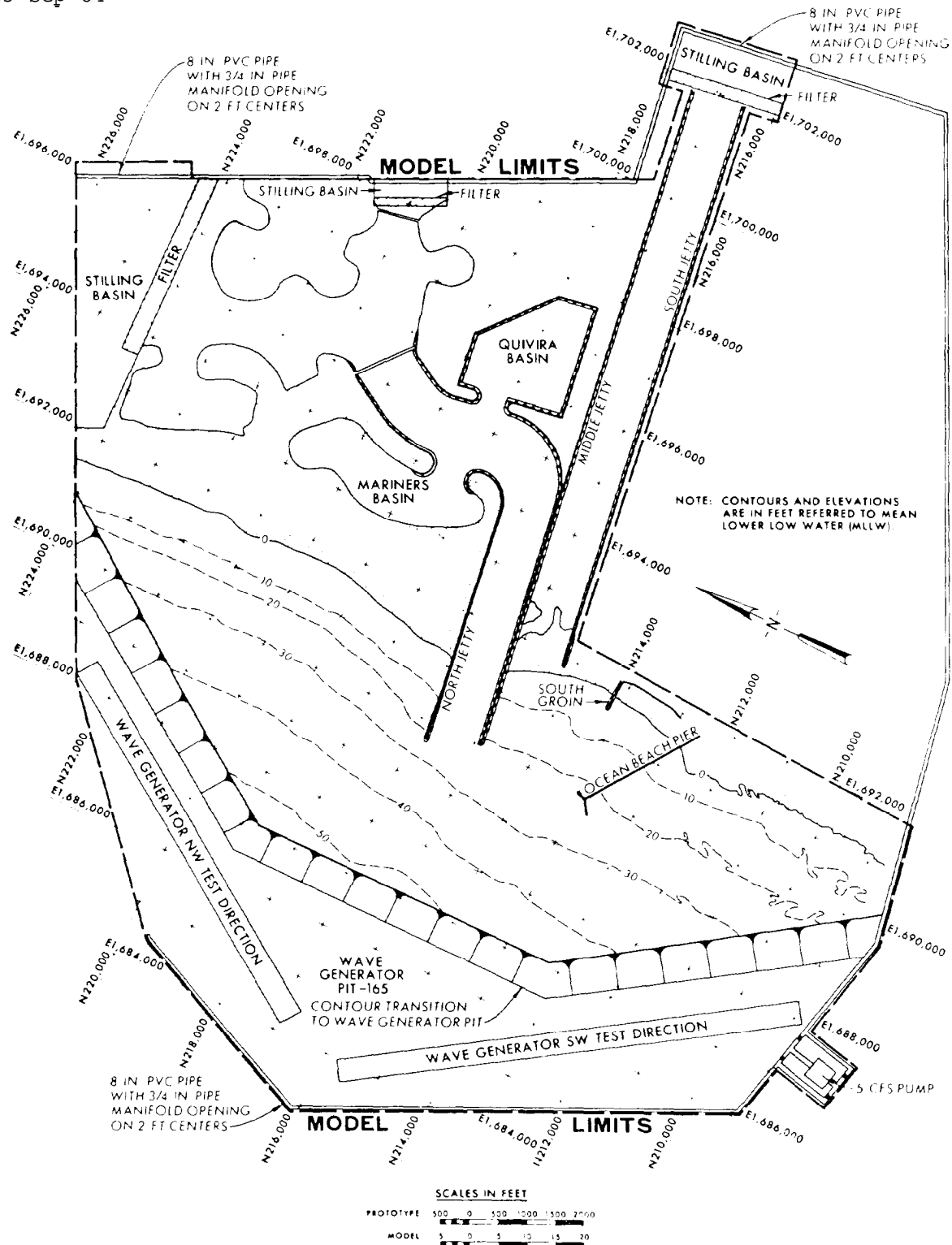


Figure A-15. Model layout, Mission Bay Harbor, California.

Still water levels (swl) selected for use during model testing were 0.0 feet, mllw (mean lower low water), +5.4 feet, mhhw (mean higher high water), and +2.7 feet used for maximum steady-state ebb and flood tidal flows. A water-circulating system was used in the model to reproduce to scale maximum steady-state ebb and flood tidal flows and various river flood flows. River discharges of 11,000-97,000 cubic feet per second were selected for testing in the model. A general view of the model is shown in Figure A-16.

(4) Tests and Results -- The Harbor.

(a) Existing conditions. Prior to tests of various improvement plans, comprehensive tests were performed for existing conditions to determine wave and current conditions inside the harbor and current and shoaling conditions outside the harbor. Existing conditions were characterized by strong long-shore currents which are redirected seaward by the north and middle jetties for moderate to large wave conditions. In general, clockwise eddies form north of the north jetty and counterclockwise eddies form south of the middle jetty. No shoaling of the harbor entrance was observed. Wave heights in the entrance channel were frequently excessive but were largely dissipated upon reaching the small boat basins. Long-period wave tests revealed substantial

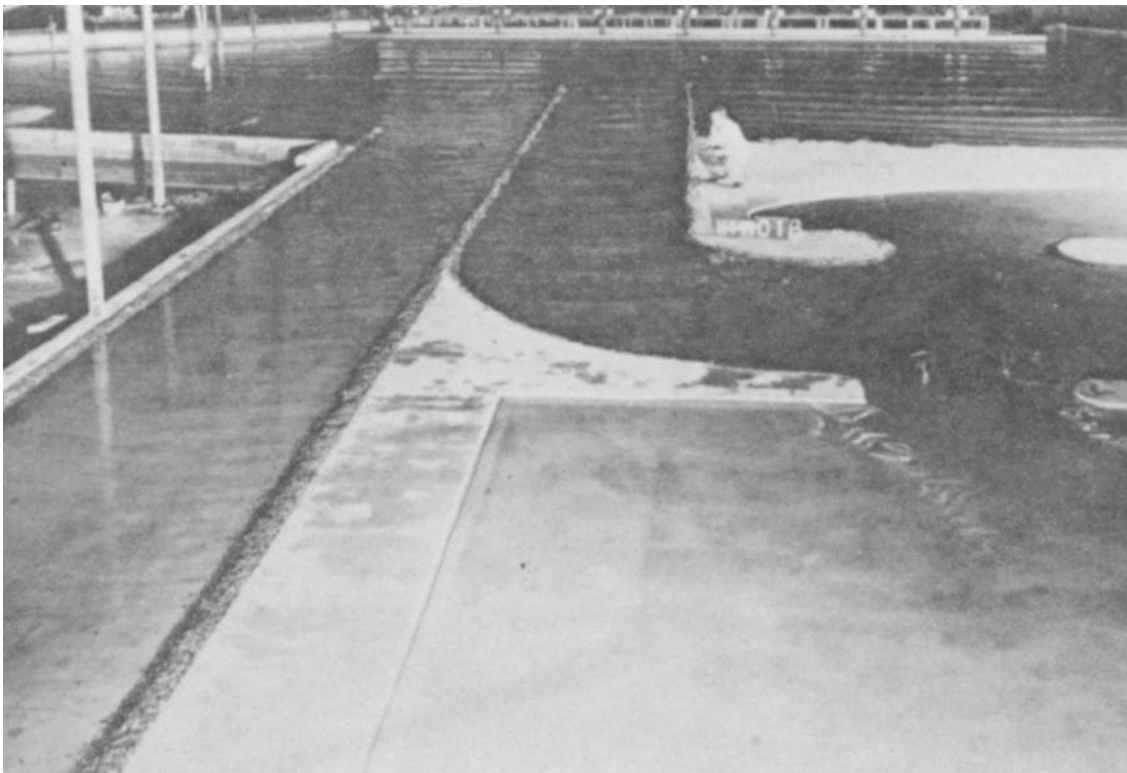


Figure A-16. General view of model, Mission Bay Harbor.

oscillations in the entrance channel and the small-boat basins for a number of incident wave periods.

(b) Improvement Plans. Tests were conducted for 30 improvement plans using various offshore breakwater designs (i.e., changes in the lengths, crest elevations, positions, and porosity of the structure). The original offshore breakwater plan for wave protection at Mission Bay Harbor was ineffective in reducing wave heights within the bay to an acceptable level. Moving the breakwater into shallower water decreased wave heights in the entrance channel to a more acceptable level, but the wave height criterion still was exceeded. It was apparent that excessive wave energy was being transmitted through the voids of the breakwater and by sealing the core of the offshore breakwater, this wave energy was largely eliminated. Of the plans tested, Plan 3G (a 1,600-foot-long breakwater at a crest elevation of 17.5 feet) provided the most effective reduction of wave energy with the least volume of rock required for construction (a reduction of 50 percent when compared with the originally proposed structure). This plan was effective, even under the most extreme conditions (i.e., removal of all revetment within the bay and an increase in swl to +7.6 feet. This plan also considerably reduced long-period waves (generally 50 percent or more) in the channel and basins. No significant shoaling of the harbor entrance was noted (Figure A-17).

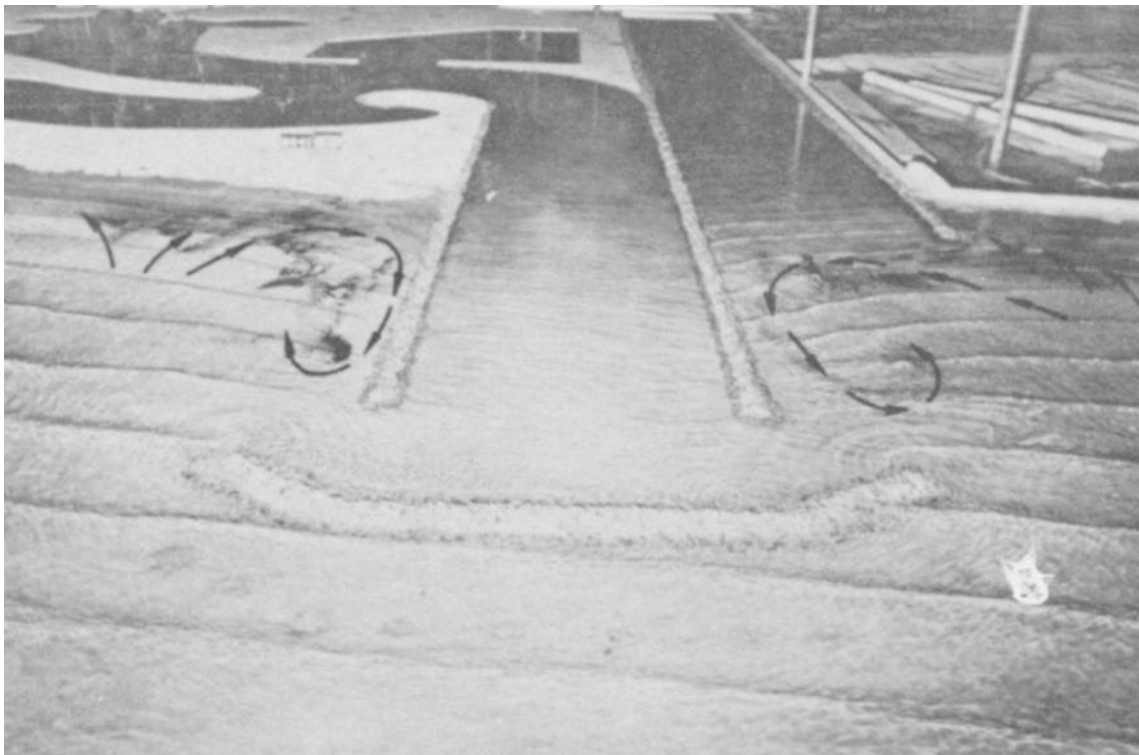


Figure A-17. Typical tracer movement for Plan 3G, Mission Bay Harbor.

25 Sep 84

(5) Tests and Results -- The River.

(a) Existing Conditions. Prior to tests of various improvement plans, comprehensive tests were conducted for existing conditions to determine the mechanisms by which sand is shoaling the river mouth and its effect on river flood flows. The river channel at project depth is prone to severe shoaling for waves from any direction, but particularly for waves from the southwest. The river channel at project depth is also quite capable of discharging the maximum flood flow tested (97,000 cfs) without causing flooding upstream. Tests of the river channel with a +10-foot-elevation sediment plug, representative of that presently blocking the river mouth, indicated a flooding hazard for the 49,000-cfs and 97,000-cfs river flows. Blowout tests also indicated potential shoaling of the south entrance to the bay (Figure A-18).

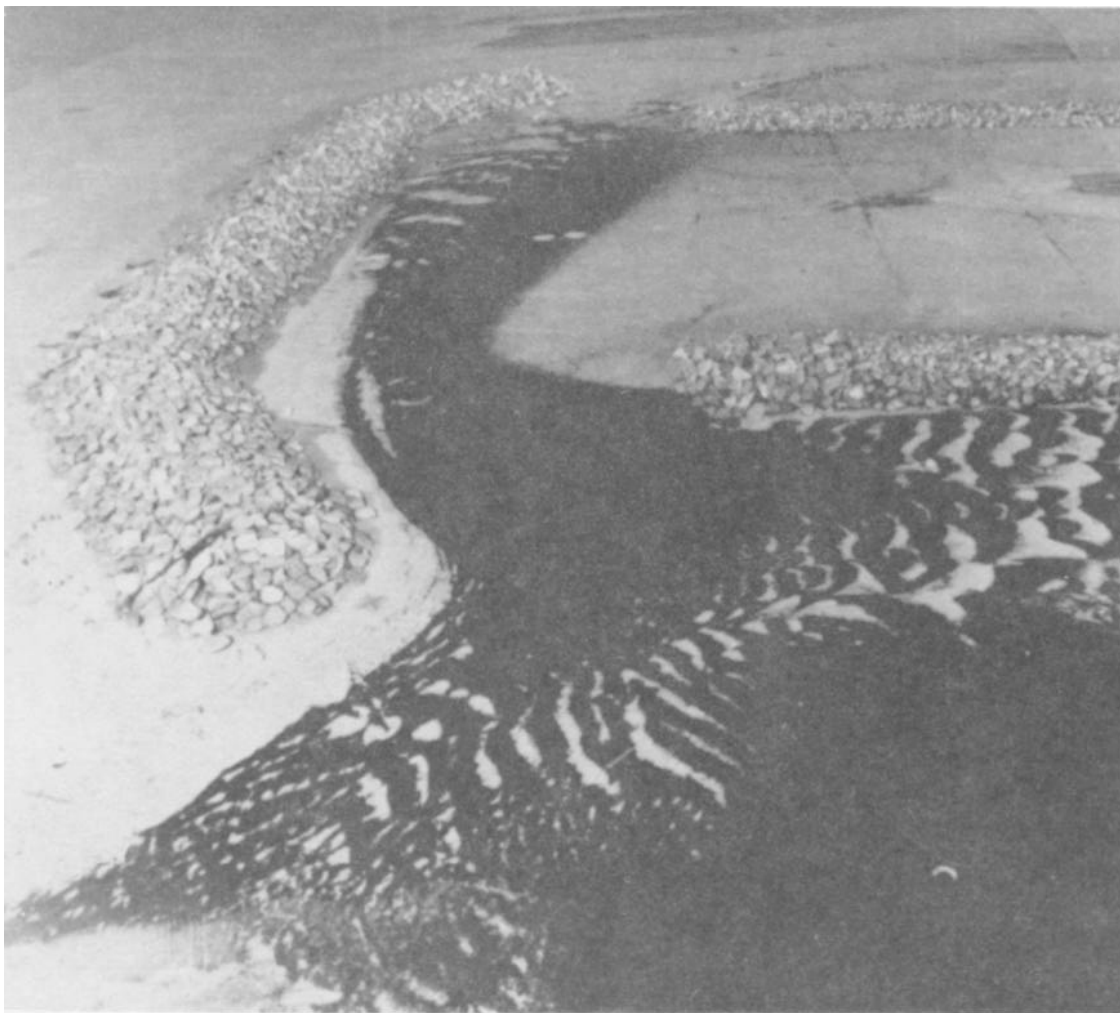


Figure A-18. Deposits at the entrance to Mission Bay Harbor as a result of blow-out tests.

(b) Improvement Plans. Tests were conducted for 29 improvement plans using various south jetty extensions, weirs, and spur jetties. Non-structural measures included incremental sediment plug removals, elevation changes and pilot channels. A reduction of the elevation of the sediment plug to +6 feet reduced the flooding hazard. However, this plan would be difficult to maintain. Removal of sections of the sand plug by dredging proved quite effective in reducing the flood hazard. Again, this plan may be difficult to maintain. Tests conducted with a weir built into the middle jetty for a +10 feet elevation sand plug showed significantly reduced water surface elevations. Of the plans tested to prevent the formation of the sand plug, a 2,373-foot-long jetty extension was effective in preventing all wave-induced river shoaling. However, because of the length of structure required, this plan would be quite expensive. A 1,273-foot-long jetty extension would eliminate channel shoaling by nearshore material. All plans involving a pilot channel cut into the sand plug worked well in preventing river flooding. A 400-foot-long spur jetty was the optimum plan tested for preventing shoaling of the south entrance to the bay during flood conditions (Figure A-19). The optimum improvement plan recommended at Mission Bay Harbor, considering wave action, shoaling, and flood control, is shown in Figure A-20.



Figure A-19. Deposits at the entrance with the 400-foot-long diversion channel installed, Mission Bay Harbor model.

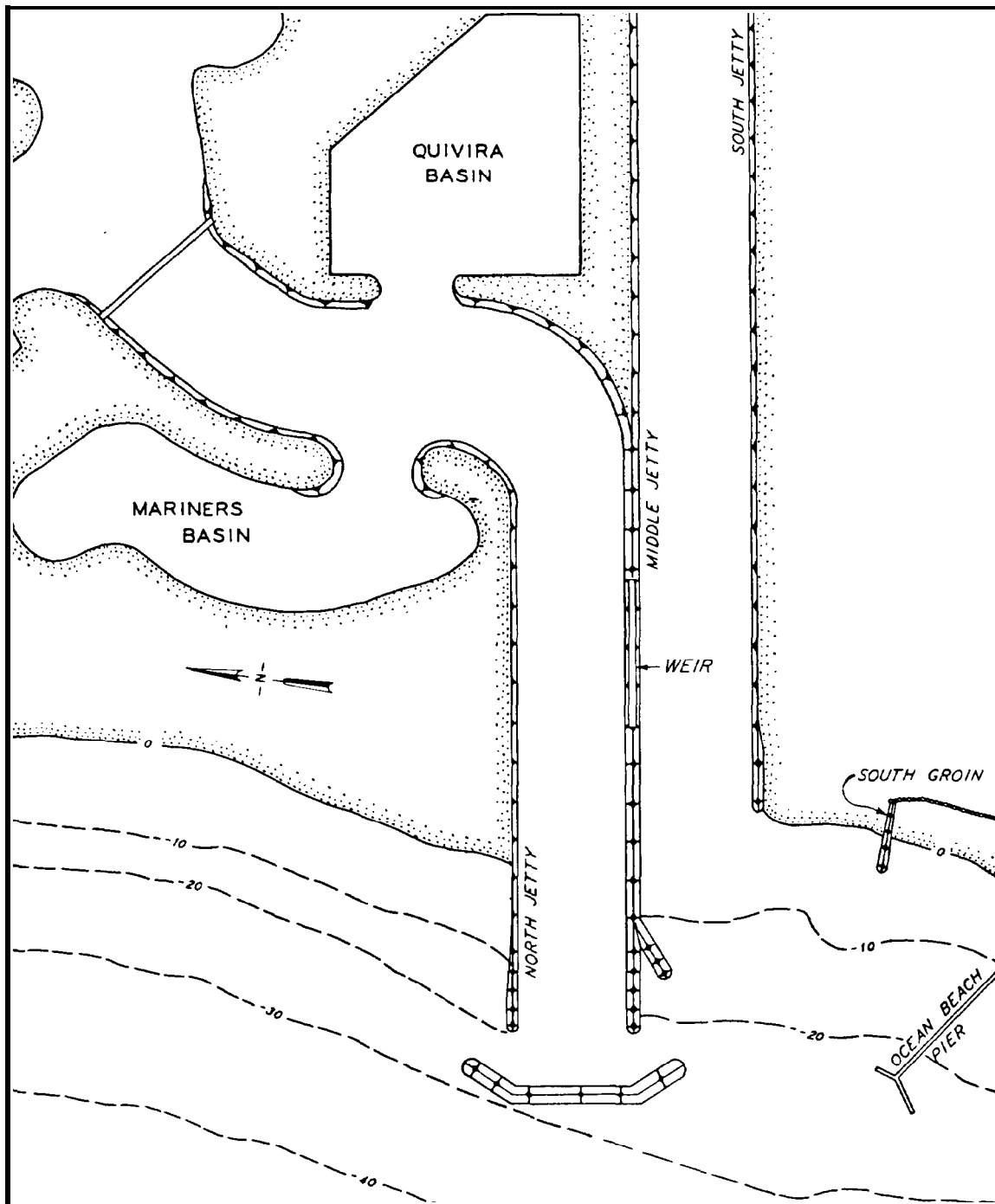


Figure A-20. Improvement plan recommended for Mission Bay Harbor.

b. Little Lake Harbor, Michigan (Seabergh and McCoy 1982).

(1) The Prototype. Little Lake Harbor is a harbor of refuge located on Lake Superior (Figure A-21) about 21 miles west of Whitefish Point and 30 miles east of Grand Marais, Michigan. The harbor is an important link in a chain of harbors along the south coast of Lake Superior which provide refuge from storms

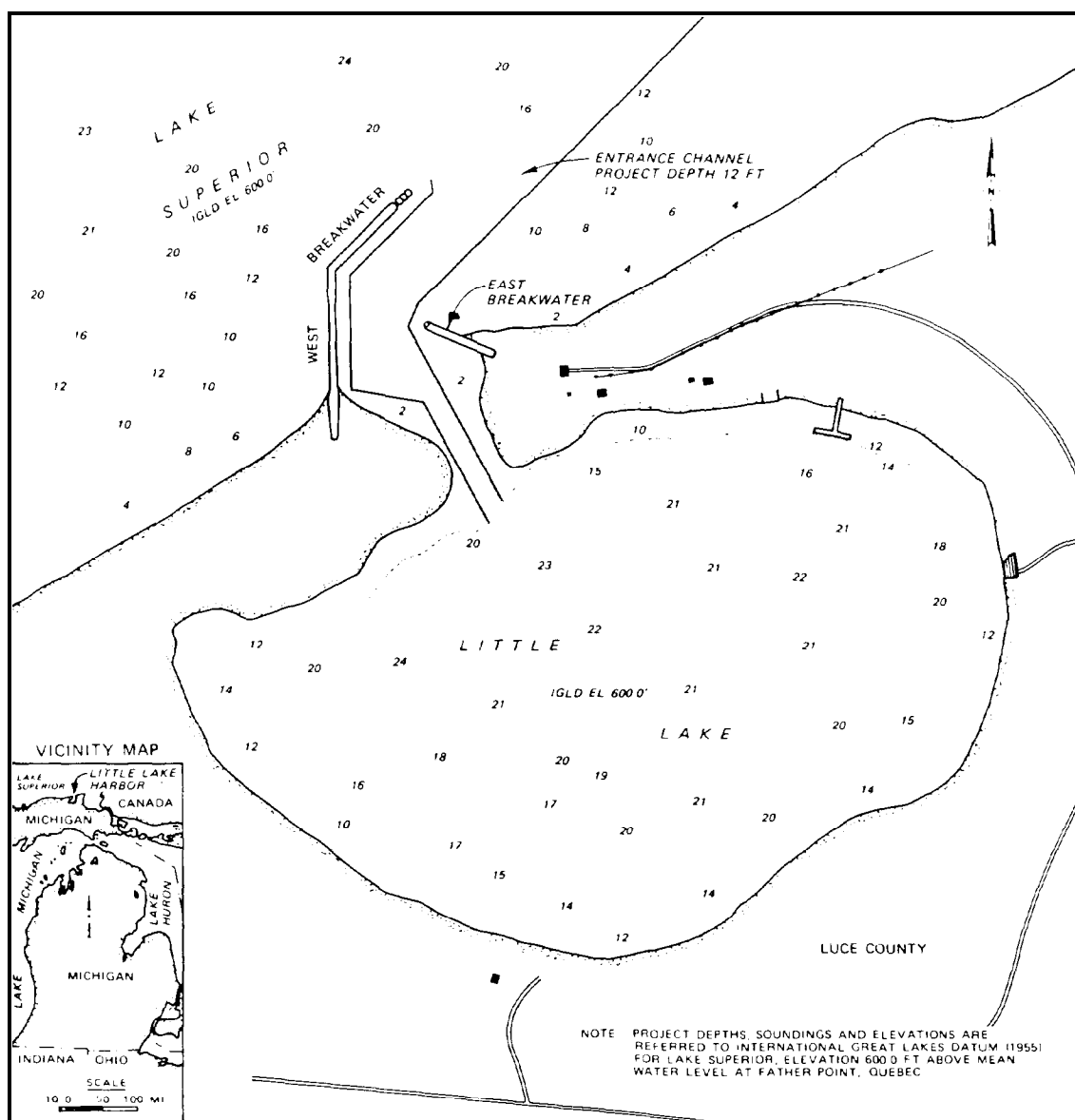


Figure A-21. Little Lake Harbor, Michigan.

for light-draft vessels. Originally, no permanent channel connected Little Lake with Lake Superior. Longshore sand movement usually closed off communication between the two bodies of water, except when sufficient rainfall raised the water in Little Lake to cause a breach in the spit. The original project (constructed between May 1962 and June 1964) consisted of two rubblemound breakwaters, with the end of each terminated by steel sheet-pile cells to provide a safe and clearly defined entrance.

(2) The Problem. Severe shoaling occurs in the Little Lake Harbor entrance channel and required dredging has averaged 33,800 cubic yards per year. All information indicates heavy shoaling on the eastern side of the channel between the two breakwaters. This heavy shoaling makes navigation to the protective harbor difficult, if not dangerous, even during relatively good weather conditions. Figure A-22 shows a fill and scour map for July 1979 to November 1979, indicating fill over nine feet in the entrance channel. The sediment entering the channel at the east jetty location can presumably be derived from both upcoast and downcoast sources. Sediments migrating from west to east around the west jetty structure under the influence of wave and wind generated currents, can move shoreward and become caught in a clockwise gyre

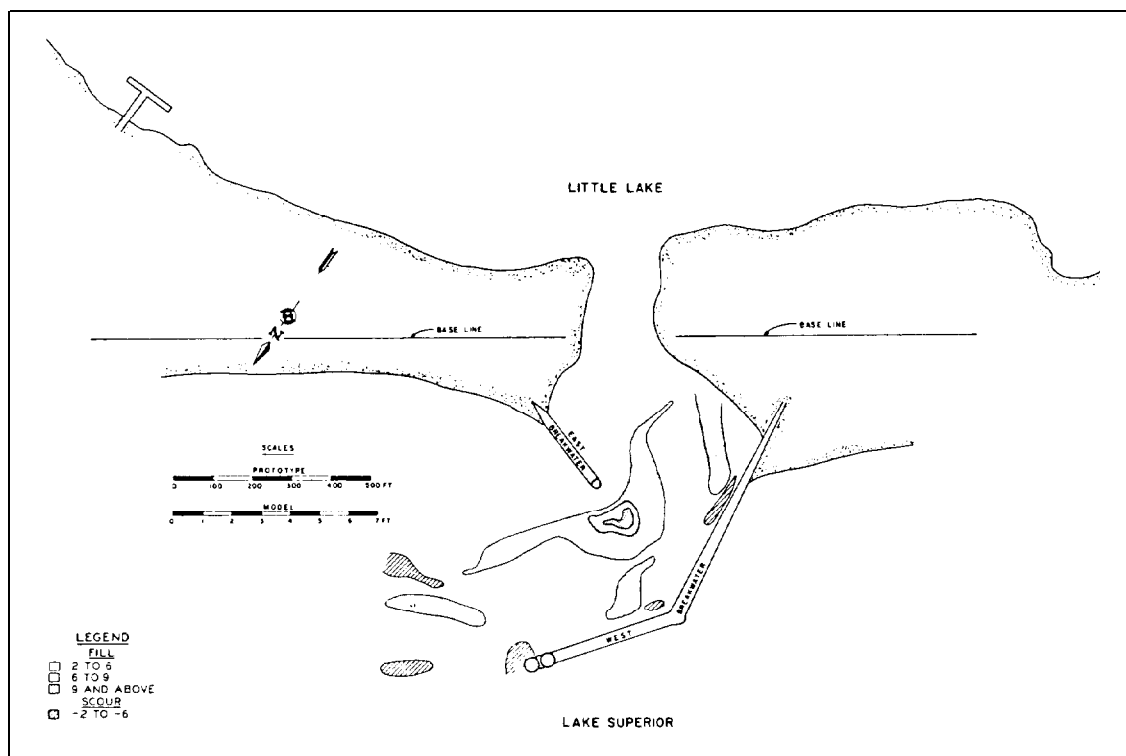


Figure A-22. Fill and scour at entrance to Little Lake Harbor, Michigan (July 1979 - November 1979).

25 Sep 84

in the lee of the west breakwater. This gyre has been observed during field work, and, combined with the action of refracted and diffracted waves, is able to move sediments toward the channel and cause shoaling. Also, any sediments which have been brought from east to west toward the entrance channel can be moved into the channel at this time, even though wave conditions are occurring from the westerly directions. When waves occur from the north to northeast, there appears to be a direct path of transport along the coast and into the channel, with an abundant supply of sand being derived from the sand cliffs that, historically, have been eroding onto the beaches east of the harbor entrance. Sediment transport through the west breakwater also has been noted, which can cause minor shoaling on the west side of the channel. Another aspect of the dynamics of the Little Lake Harbor area relates to the occurrence of seiche activity in Lake Superior and the generation of currents through the Little Lake Harbor entrance channel and bay. Seiche currents of up to 5 fps can occur and influence sediment movement in the area by augmenting the gyre circulation patterns.

(3) The Model, Prototype Data, and Test Conditions. The Little Lake Harbor model was constructed in a concrete basin 150 feet long by 120 feet wide by 2 feet deep to a 1:75 (undistorted) scale. About one mile of beachline both upcoast and downcoast of the harbor was modeled, as seen in Figure A-23. Prototype water level gages were installed in the sheltered bay and in the open lake to evaluate seiche activity. From these data it was determined that the most frequent seiche period was about 0.5 hour, which coincided with the resonant Helmholtz period. This type of oscillation is characterized by the bay level rising uniformly, with the inlet channel water mass and the rise and fall of the bay acting together as a spring-mass system. Waves selected for testing for the base conditions are shown in Table A-4.

(4) Tests and Results. Testing performed for the model study primarily involved tracer tests, in which sediment tracer material (crushed coal) was injected into the surf and nearshore zones in the vicinity of the harbor for a given wave condition. Each test was run for a sufficient length of time to allow tracer movement and deposition patterns to develop, and a photograph then was taken to illustrate test results. Also for given wave conditions, a pattern of movement of the water mass in the nearshore zone adjacent to the harbor was determined using dye. Point velocities at selected locations were measured by timing the movement of a patch of dye over a known distance, and wave heights were measured at selected locations for various wave conditions. For some tracer tests, seiche oscillations were reproduced in addition to the wave field. Also, seich oscillations were reproduced and velocity measurements were made with current meters in the entrance channel region. Surface current photographs also were obtained during seiche reproduction by making a 4-sec time exposure of the water surface covered with Styrofoam floats. The testing program followed this sequence: base tests, using existing 1979 conditions with the channel dredged; initial plan testing, in which five proposed plans were examined; additional plan testing, in which plans were refined

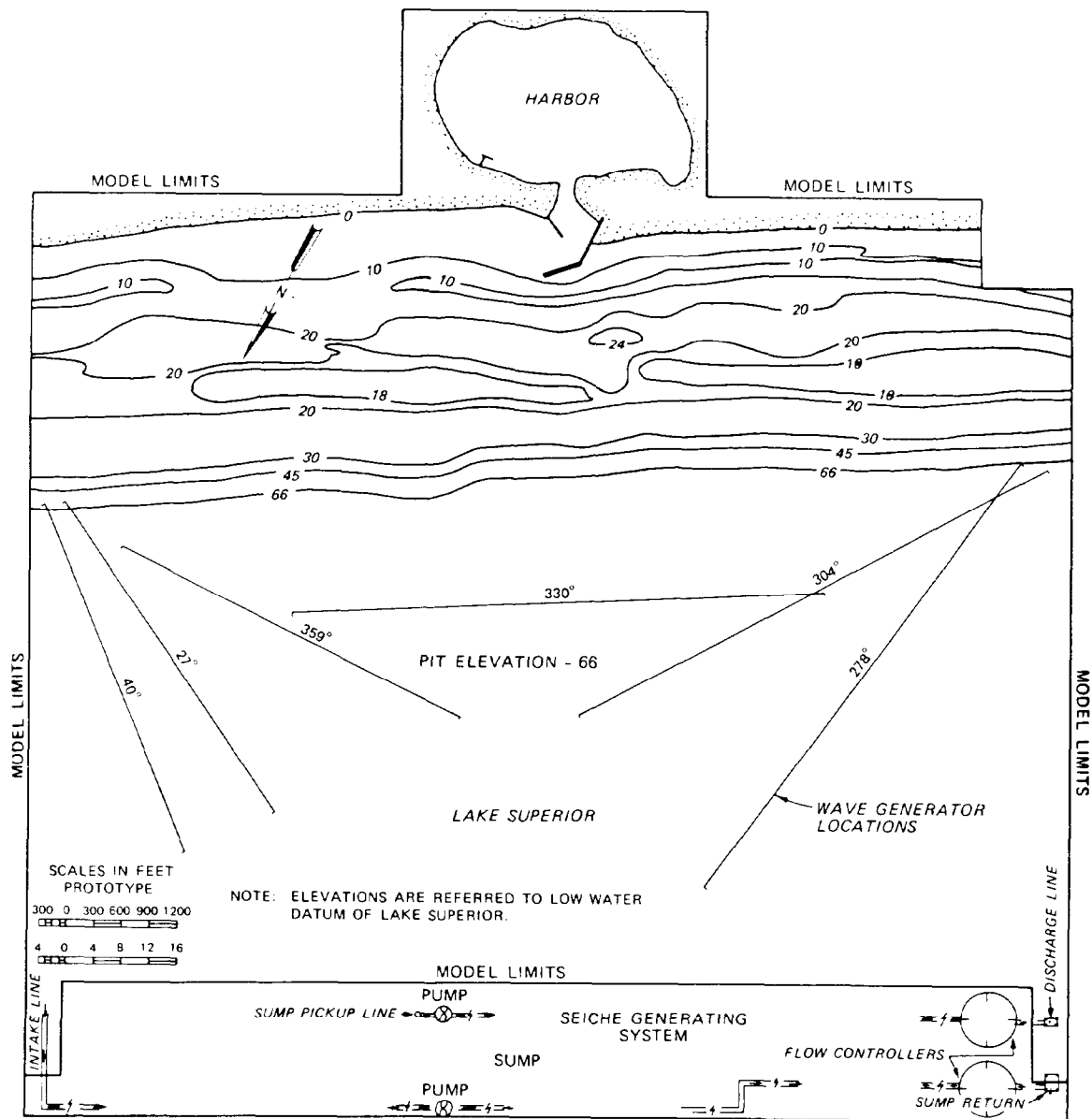


Figure A-23. Model layout, Little Lake Harbor, Michigan.

TABLE A-4

Test Waves Used in the Little Harbor Harbor Model
(Resio and Vincent, 1978)

<u>Deepwater Wave Direction</u>	<u>Shallow Water Wave Test Direction</u>	<u>Test Wave</u>	
		<u>Period (sec)</u>	<u>Height (ft)</u>
46.5	40	5	4
		7	10
		9	16
30	27	5	4, 7
		7	5, 10
		9	8, 16
0	359	5	4, 7
		7	12
		9	10, 21
330	330	5	4, 7
		7	6, 12
		9	10, 21
301	304	5	4, 7
		7	5, 10
		9	8, 17
272	278	5	4, 7
		7	5, 10
		9	8, 17

based on what was learned from the initial plan testing; and final plan testing, where the final plan was examined comprehensively for additional test conditions. Base tests indicated the mechanisms by which the channel shoaled with sediment moving into the channel along the short east breakwater (Figure A-24) regardless of direction. A variety of plans were examined, with the best



Figure A-24. Tracer deposits in the Little Lake Harbor model for base tests.

plan seen in Figure A-25. This plan provided for good natural bypassing of sediments for larger wave conditions. The gap between the new east structure and the shore should eventually close with a natural accumulation of sand and was seen to do so in model tests (Figure A-26).

A-9. Harbors Built Inside a River/Stream Mouth. Numerous rivers and streams empty into the oceans and Great Lakes. Many of these locations are used as small-boat harbor sites. Rogue River Harbor, Oregon, situated on the Pacific coast, and Cattaraugus Creek Harbor, New York, located on Lake Erie, were selected as representative harbors under this classification and are discussed below.

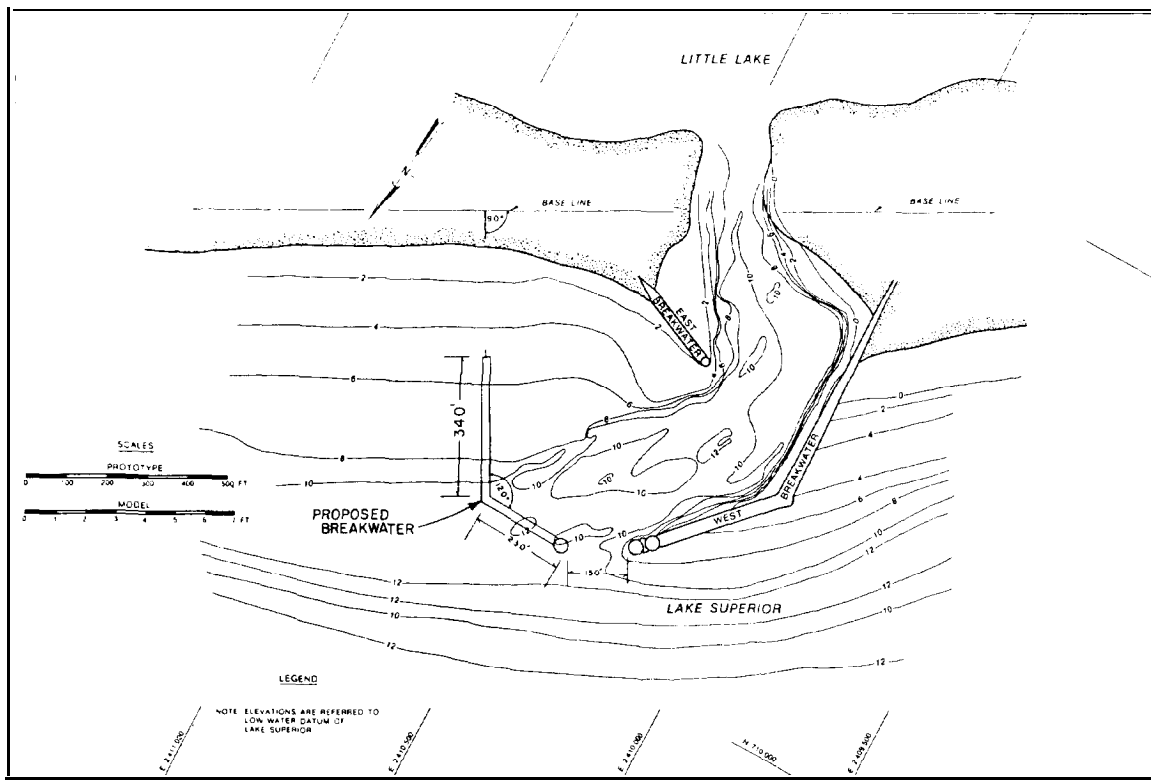


Figure A-25. Optimum improvement plan, Little Lake Harbor.

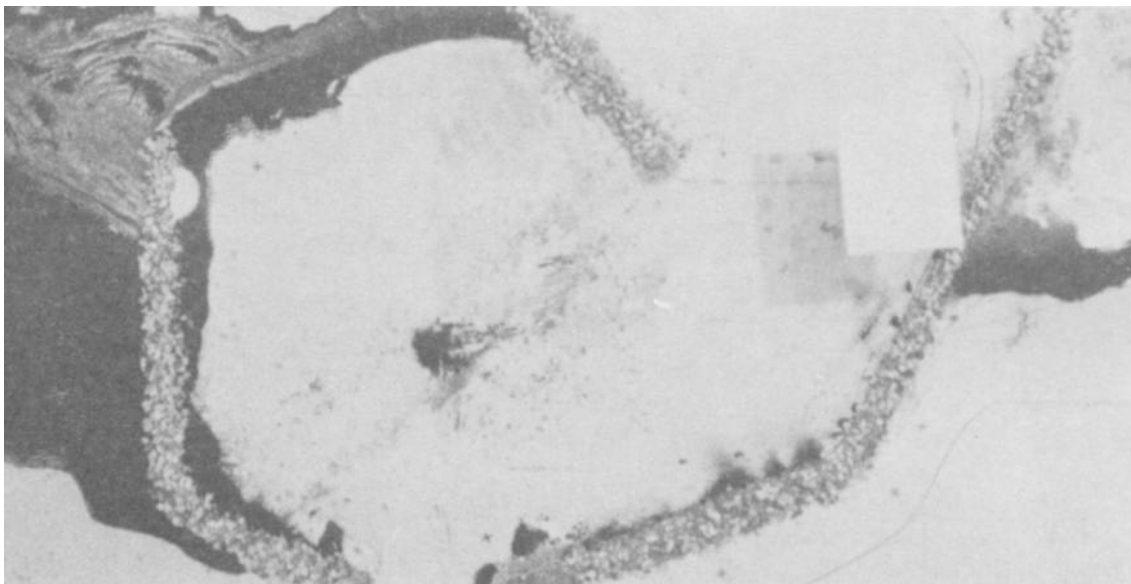


Figure A-26. Natural accumulation of sand closing gap between the new east structure and shore, Little Lake Harbor model.

a. Rogue River Harbor, Oregon (Bottin 1982).

(1) The Prototype. The Rogue River originates in the Cascade Mountain Range and flows generally westerly entering the Pacific Ocean on the Oregon coast approximately 30 miles north of the California border (Figure A-27). The river is about 180 miles long and drains an area of approximately 5,100 square miles (CTH 1970). The principal communities at the mouth are Gold Beach and Wedderburn, located on the south and north banks, respectively. These areas are developed for resort and recreational usage. Prior to improvements, the river channel at the mouth meandered between two sand spits and was seldom less than 200-feet wide at low water. Controlling depths over the entrance bar ranged from two feet in late summer to nine feet in winter. The River and Harbor Act of 1954 provided for the construction of parallel jetties spaced approximately 1000 feet apart at the mouth of the river. In 1971 and 1972, the Port of Gold Beach constructed a breakwater that extended from a point on the south bank (about 1000 feet above the U. S. 101 Highway bridge) downstream to the south jetty. A gap was left in the breakwater to provide access to harbor facilities.

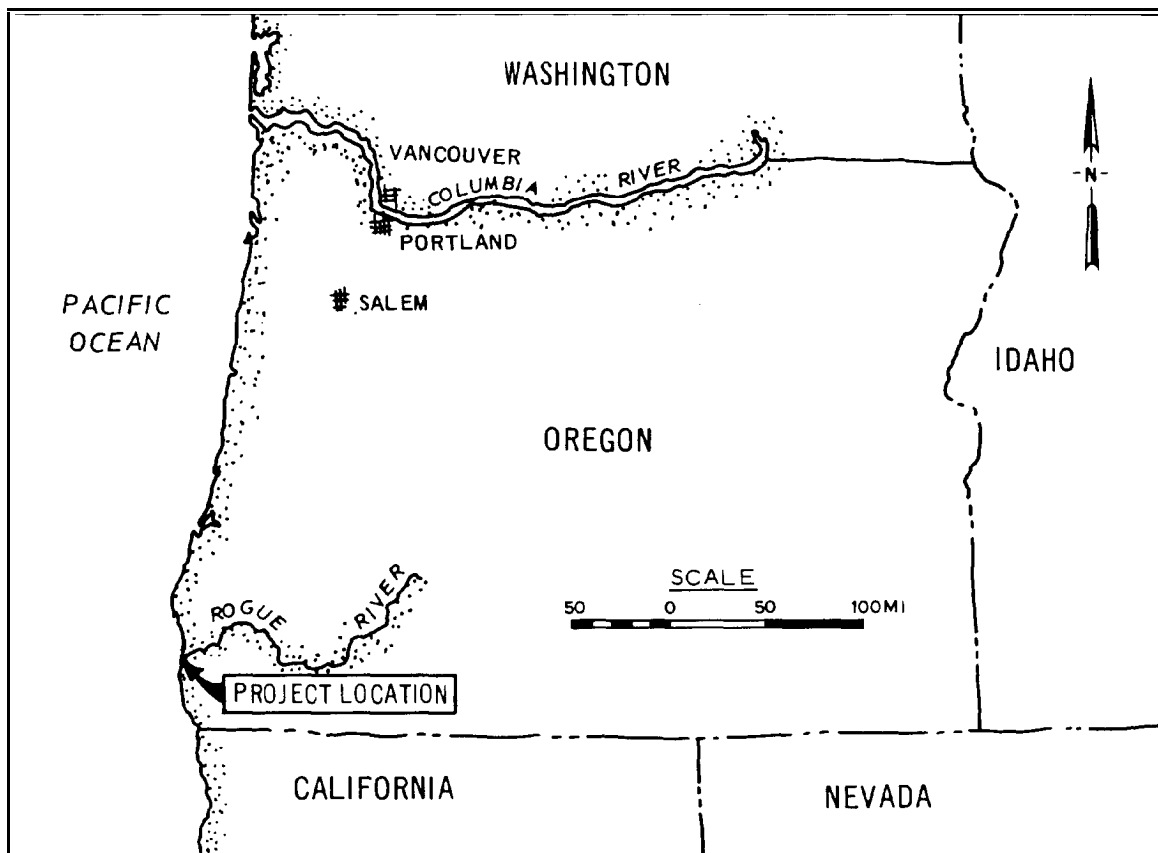


Figure A-27. Project location, Rogue River, Oregon.

(2) The Problem. Every year a persistent shoaling problem exists between the Rogue River jetties. This shoal extends upstream along the inside of the south jetty and across the harbor access channel (Figure A-28). This condition makes maintenance dredging difficult and blocks navigation channels, thus restricting vessel traffic between the ocean and port facilities. Rapid summer-time shoaling occurs (when river flows are normally low) during the peak boating and salmon fishing seasons, causing unpredictable and hazardous entrance conditions. Authorized channel dimensions cannot be maintained by dredging due to the rapid shoaling rate. Annual maintenance dredging costs in excess of \$100,000 are expended with large backlogs of dredging to be done.

(3) The Model and Test Conditions. A physical model investigation was conducted to study shoaling, wave, current, and riverflow conditions in the lower reaches of the Rogue River for existing conditions and proposed improvements. The Rogue River Harbor model (Figure A-29) was constructed to an undistorted linear scale of 1:100, model to prototype. Test waves used in the model study with periods ranging from 5 to 17 seconds and heights ranging from 7 to 29 feet are shown in Table A-5. A water circulating system was used to

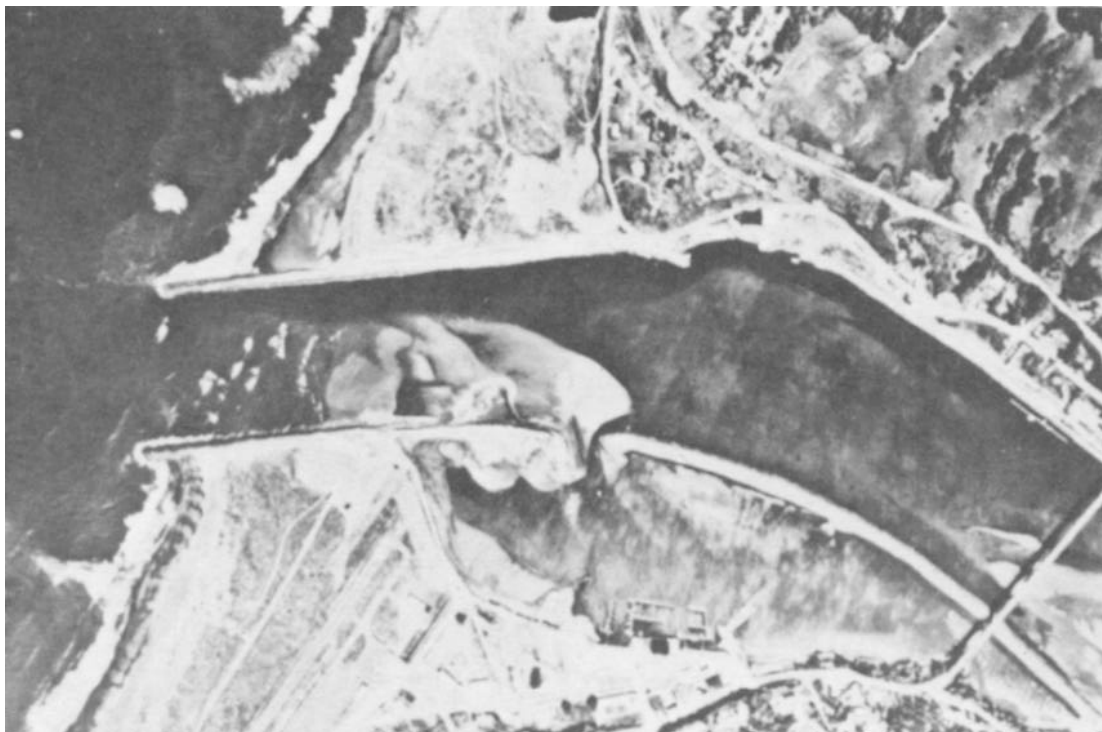


Figure A-28. Aerial photograph of Rogue River mouth.

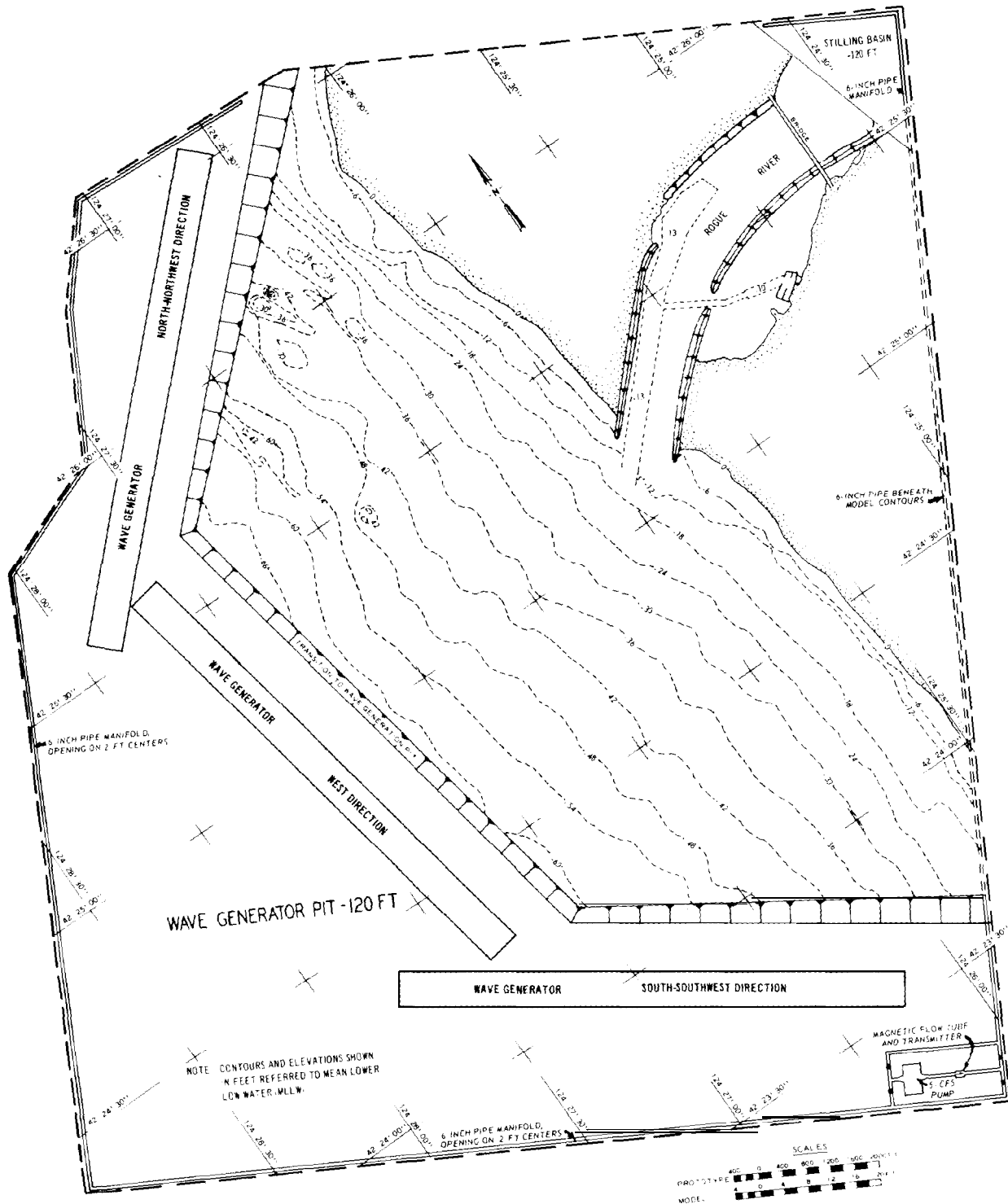


Figure A-29. Model layout, Rogue River, Oregon.

TABLE A-5

Test Waves Used in the Rogue River Harbor Model (NMC, 1960)
(SMO, 1976) (FNWC, 1977)

Deepwater Direction	Selected Test Waves	
	Period (sec)	Height (ft)*
North-northwest	5	7, 12**
	7	7, 12, 20**
	9	7, 12, 17, 27
	11	7, 12, 19
	13	7, 13, 21
	15	7, 11, 17
	17	7, 11
West	5	7, 12**
	7	7, 12, 20**
	9	7, 12, 23, 31
	11	7, 12, 23, 31
	13	7, 12, 21, 29
	15	7, 12, 21, 29
	17	7, 12, 17
Southwest	5	7, 12**
	7	7, 12, 20**
	9	7, 13, 21, 27
	11	7, 13, 21, 29
	13	7, 13, 21, 27
	15	7, 12, 17, 25
	17	7, 12, 18
South-southwest	5	7, 12**
	7	7, 12, 20**
	9	7, 12, 17, 27
	11	7, 12, 17, 27
	13	7, 12, 21
	15	7, 12, 23
	17	7, 12, 18

* Wave heights shown are shallow-water values (adjusted as a result of re-fraction-shoaling analysis).

** Steepness limited waves.

reproduce steady-state flows that corresponded to maximum flood and ebb tidal flows or various river discharges. River discharges ranging from 50,000 to 350,000 cfs were reproduced in the model. A coal tracer material was used in the model to qualitatively determine the degree of shoaling at the river mouth. Still-water levels of 0.0 foot (mllw), +1.5 feet (maximum ebb), +4.3 feet (maximum flood), and +6.7 feet (mhhw) were used during model testing. An automated data acquisition and control system was used to secure wave height data, and water-surface profiles for various river discharges were determined by recording elevation changes on point gages located at various stations in the river. A general view of the model is shown in Figure A-30.

(4) Tests and Results.

(a) Existing Conditions. Prior to tests of the various improvement plans, comprehensive tests were conducted for existing conditions. Wave-height data, wave-induced current patterns and magnitudes, shoaling patterns, and wave pattern photographs were obtained for representative test waves from the four selected test directions. Water-surface elevations and river current velocities also were obtained for the various river discharges. During the conduct

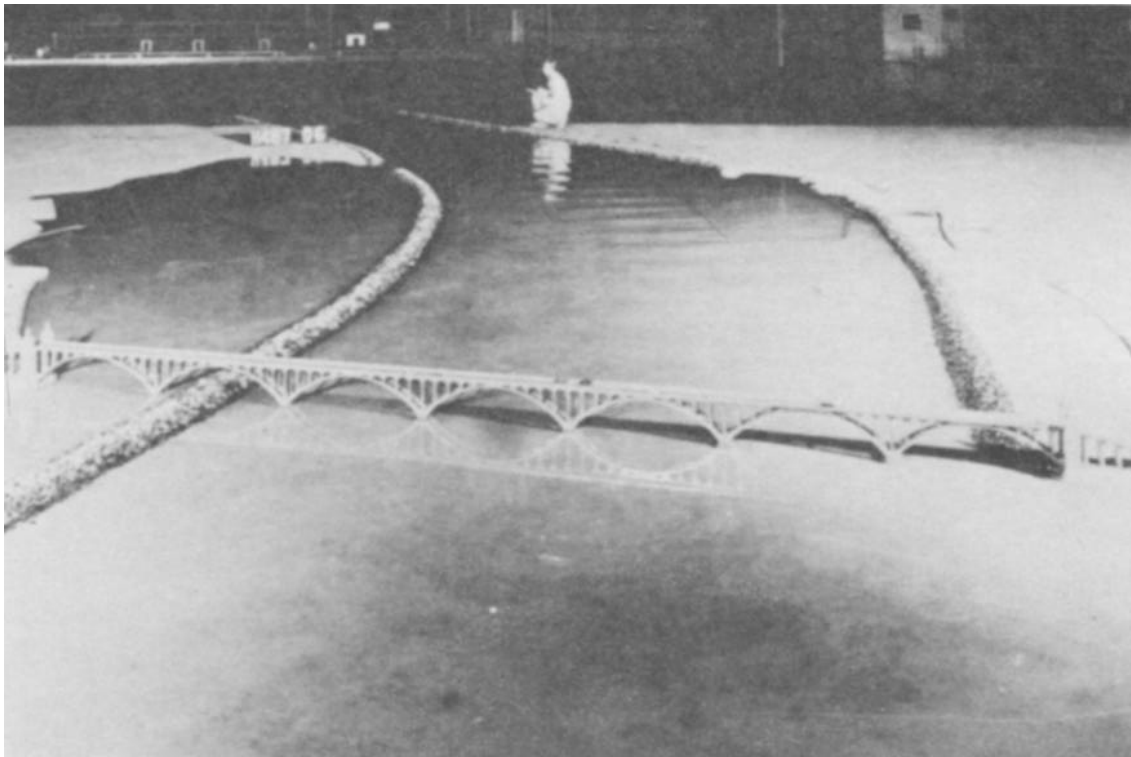


Figure A-30. General view of model, Rogue River, Oregon.

25 Sep 84

of shoaling tests, tracer material was introduced into the model south of the south jetty and north of the north jetty to represent sediment from those shorelines, respectively. In addition, tracer was introduced seaward of the river mouth to represent sediment washed out of the river and deposited by various discharges. Shoaling tests conducted for existing conditions indicated that shoaling would occur in the lower reaches of the river for various test waves for each wave direction. Generally, material deposited in the southern portion of the river adjacent to the south jetty. Under constant wave attack, this material would congregate against the south jetty and migrate upstream across the entrance to the small-boat harbor (Figure A-31) forming a shoal similar to that of the prototype. It was also noted that, when the shoal is present, rough and turbulent wave conditions exist in the entrance (due to waves breaking on the shoal) and higher than normal river stages and river-current velocities may result for various discharges (since the shoal interferes with the passage of flood flows). When the shoal is not present, increased wave heights can be expected upstream of the small-boat harbor entrance.

(b) Improvement Plans. Model tests were conducted for 58 variations in the design elements of three basic remedial improvement plans. Dikes installed within the existing entrance, extensions of the existing jetties, and

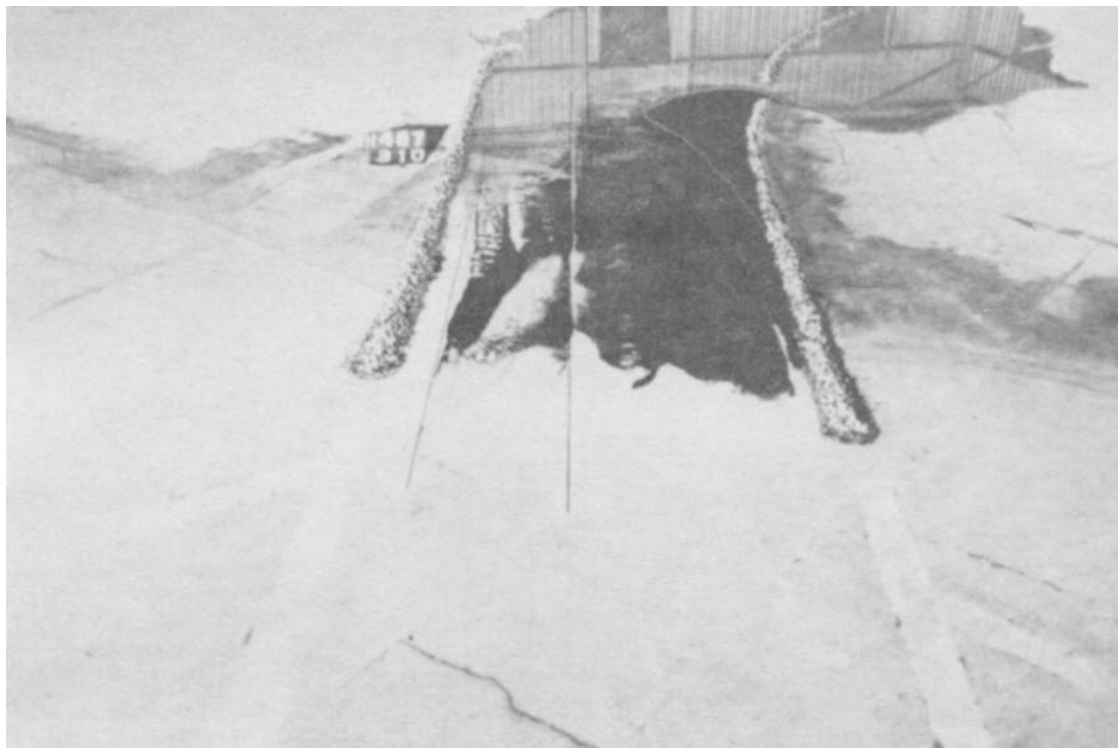


Figure A-31. Shoal formed in the river entrance for existing conditions, Rogue River, Oregon.

an alternate harbor entrance were tested. Wave-height tests, wave-induced current patterns and magnitudes, wave patterns, water-surface elevations, river current Velocities, and/or shoaling tests were conducted for the various improvement plans. The first series of test plans included the installation of dikes within the existing entrance. Both timber-pile and rubble-mound dikes were tested. Test results indicated shoaling of the small-boat harbor entrance would occur for test plans with the timber-pile dikes installed. The rubble-mound dike configuration, however, intercepted the movement of tracer material and prevented it from shoaling the harbor entrance. Water-surface elevations obtained for the dike plans indicated that river stages would increase, when compared to those for existing conditions, and potentially may contribute to flood problems. The installation of a weir section in the existing north jetty and a conveyance channel on the north overbank reduced river stages upstream by less than one foot and therefore was not successful in decreasing water-surface profiles to desired levels. The next series of test plans involved extensions of the existing jetties. One plan entailed extending the jetties on their original alignment, another involved orienting the extensions toward the west (on an azimuth of $S81^{\circ}41'30''W$) and still another consisted of orienting the extensions toward the south (on an azimuth of $S16^{\circ}23'22''W$). Test results, with the extensions on the original jetty alignments, indicated that sediments from the river would form a shoal in the entrance adjacent to the south jetty that would extend upstream across the small-boat harbor entrance similar to existing conditions. With the test plans involving jetty extensions oriented toward the west, sediment from the river would form shoals in the river entrance but would not extend upstream to the small-boat basin entrance. With the test plans involving jetty extensions to the south, sediment from the river would result in a shoal along the south jetty extension, extending northerly into the entrance. The shoals formed in the river entrance for all three jetty extension plans were due to sediment being washed out of the river and migrating back in, since each plan series was modified to provide shoaling protection from sediment on the north and south shorelines. The last series of test plans involved a new entrance south of the existing river mouth. Test results indicated that this new jetty configuration (Figure A-32) would provide shoaling protection for the new entrance from sediment on the north and south shorelines and sediment deposited seaward of the river entrance by various discharges. In addition, this plan would provide wave protection to the small-craft harbor with maximum wave heights less than one foot.

b. Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975).

(1) The Prototype. Cattaraugus Creek drains an area of about 580 square miles on the south shore of Lake Erie. The creek is approximately 70 miles long and flows generally westward, entering the lake about 24 miles southwest of Buffalo Harbor, New York (Figure A-33). For about 17 miles near its mouth, topography of the creek valley is generally flat, with a valley bottom width of 1 to 2 miles. The south side of the creek borders Hanover, Chautauqua County, New York, and the north side borders Brant, Erie County. The



Figure A-32. Wave patterns for the new entrance and jetty configuration installed south of the existing river mouth, Rogue River, Oregon.

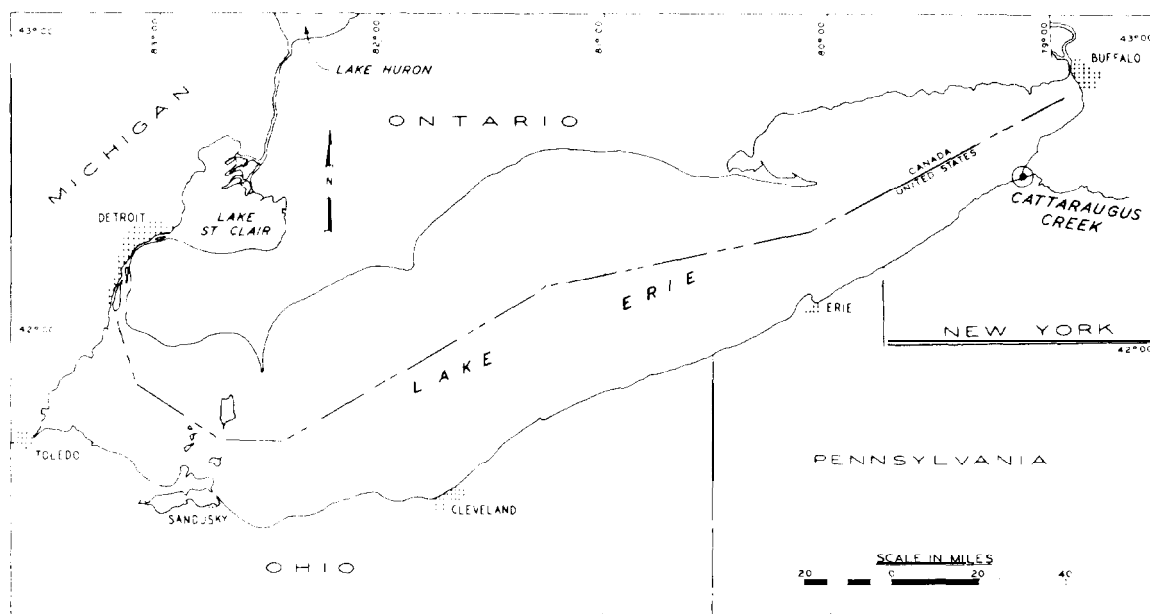


Figure A-33. Project location, Cattaraugus Creek Harbor, New York.

25 Sep 84

Cattaraugus Reservation of The Seneca Nation of New York Indians occupies the entire northern side of the creek within the study area. The present harbor encompasses the lower 3/4 mile of the creek where over 400 boats are permanently based at local marinas. The economy of the immediate area is primarily recreational and most of the residences are summer cottages. Cattaraugus Creek attracts patrons from well beyond the limits of the local communities because of its location near good recreational fishing areas in Lake Erie and the scarcity of similar facilities to meet the increasing demands of small-boat owners. Proposed improvements at Cattaraugus Creek included dredging of an entrance channel and interior channel in the lower reaches of the creek to accommodate the movements of small-craft and installation of breakwaters at the creek mouth to provide wave and shoaling protection.

(2) The Problem. Flooding occurs almost every year along the lower reaches of Cattaraugus Creek during late winter and early spring, when the creek is swollen by melting snow and spring rains, and frequently results in damages in the summer resort area of Sunset Bay, the town of Hanover, and the summer resort area in the Cattaraugus Indian Reservation. This flooding is partially due to the limited capacity of the existing creek channel, but the major contributing factor is the presence of a restrictive sand and gravel bar at the creek mouth (Figure A-34). This bar, formed mainly by littoral drift

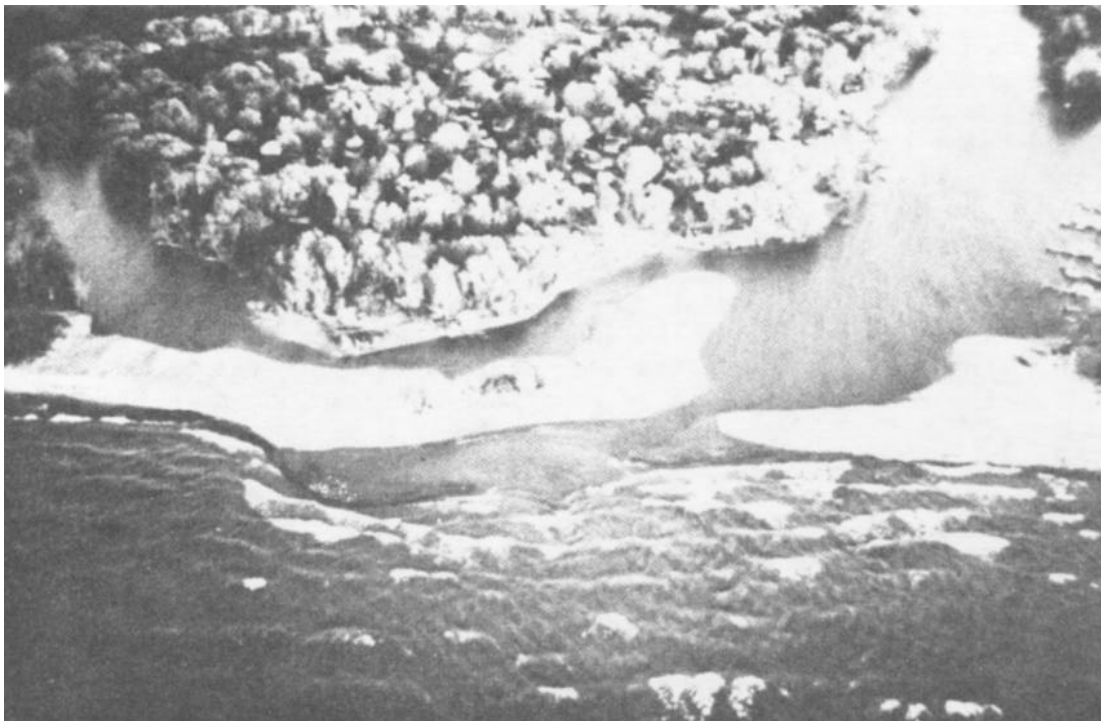


Figure A-34. Aerial photograph of Cattaraugus Creek mouth prior to improvements.

25 Sep 84

due to wave action, at times virtually closes the outlet and provides a natural barrier, encouraging the formation of ice jams. These ice jams result in significantly higher stages and damages than those caused by discharge only. Thus, considerable damages occasionally occur with only moderate creek discharges. Navigational difficulties are also experienced at the mouth of the creek due to the shallow depths and the constant shifting of the bar across the entrance. Boats leaving the harbor under favorable weather conditions find it difficult and dangerous to return over the shallow bar if wave action increases while the boats are in the open lake. Even experienced boaters who are familiar with the harbor frequently encounter groundings, which damage propellers, shafts, and rudders of the boats involved. At the end of the peak navigation season, when lake levels are normally low, the outlet is almost completely closed to navigation. In summary, improvements are needed at the entrance and lower reaches of the creek to stabilize the mouth, to provide adequate channel capacity for passage of flood flows and ice, to provide adequate depths throughout the navigation season for use of small craft, and to provide wave protection for boats moored in the harbor.

(3) The Model and Test Conditions. A physical model investigation was conducted to study shoaling, wave action, flood and ice flow conditions at the harbor entrance and lower reaches of the creek for existing conditions, and proposed improvement plans. The Cattaraugus Creek Harbor Model (Figure A-35) was constructed to an undistorted linear scale of 1:75, model to prototype. Test waves used during model operation with periods ranging from 6 to 9 seconds and heights ranging from 4 to 14 feet are shown in Table A-6. A water circulating system was used to reproduce steady-state flows through the creek channel and outer harbor area that corresponded to prototype discharges ranging from 5,000 to 57,900 cfs. Crushed coal and granulated nylon materials were used in the model to qualitatively determine the degree of shoaling at the creek mouth, and a low-density polyethylene sheet material (recommended by the Cold Regions Research and Engineering Laboratory, Corps of Engineers) was used to simulate ice in the model. Still-water levels of +3.0 and +6.8 feet were used during model testing. An automated data acquisition and control system was used to secure wave heights and water-surface elevations at selected locations in the model. A general view of the model is shown in Figure A-36.

(4) Tests and Results.

(a) Existing Conditions and Base Test. Prior to tests of various improvement plans, comprehensive tests were conducted for existing conditions and a base test. The base test entailed the proposed dredged channels with no breakwaters and was used as a base to evaluate the effectiveness of the various breakwater configurations. Existing conditions were simulated by filling the dredged channel with sand in the entrance and lower reaches of the creek. Shoaling patterns and ice flows were obtained for existing conditions, while wave height data, and wave-induced current patterns and magnitudes, water-surface elevations, and creek current velocities were secured for base test for representative test conditions. Shoaling tests conducted for existing

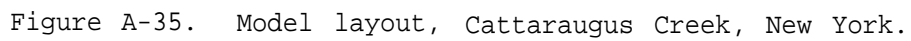


TABLE A-4

Test Waves Used in the Cattaraugus Creek Harbor Model
(Saville 1953, Bretschneider 1970)

<u>Deepwater Direction</u>	<u>Shallow-water Direction</u>	<u>Selected Test Waves</u>	
		<u>Period (sec)</u>	<u>Height (ft)*</u>
Northwest	N 40° W	6	5
		6	9
West	N 79° W	6	7
		6	14
		9	7
		9	14
West-southwest	** --	6	4

* Wave heights shown are shallow-water values (adjusted as a result of refraction-shoaling analysis.

** Locally generated wave.



Figure A-36. General view of model, Cattaraugus Creek, New York.

conditions resulted in spits forming across the creek mouth. Various creek discharges shifted these spits lakeward. Results of these tests generally indicated that the model accurately reproduced the sediment patterns observed in the prototype. For existing conditions, simulated ice material was placed in the lower reaches of the creek upstream of the spit across the river entrance and subjected to creek discharges of 5,000 and 10,000 cfs. Ice jams formed at the mouth for each discharge and overbank flooding was observed. The 10,000-cfs discharge eventually eroded the spit and the ice material moved into the lake. Wave height data obtained for base test (no breakwaters) revealed that protection from storm waves is required for small boats moored in the creek during high lake levels. Wave heights exceeded the established wave-height criteria of 2.5 feet at the creek mouth and 0.5 feet in the lower reaches of the creek.

(b) Improvement Plans. Model tests were conducted for nine variations in the design elements of two basic breakwater configurations. The first breakwater configuration (initially proposed improvement plan) consisted of a navigation opening and entrance channel oriented toward the west, and the second configuration entailed a navigation opening and entrance channel oriented toward the northeast. Variations involved changes in the lengths and alignments of the structures and the type of structures used. Test results for the breakwater configuration oriented toward the west revealed favorable wave conditions in the harbor; however, tracer tests resulted in sediment deposits in the entrance for test waves from all directions. For all the improvement plans, tracer material was introduced into the model east and west of the breakwaters to represent sediment from those shorelines, respectively, and lakeward of the entrance to represent sediment deposits from the creek for a 10,000-cfs discharge. Since the predominant direction of littoral drift at and near the mouth of Cattaraugus Creek was from southwest to northeast, the initially proposed breakwater configuration (entrance oriented toward the west) was not considered feasible and was abandoned. Modifications were made to the second breakwater configuration (entrance oriented toward the northeast) until a plan was developed that provided optimum shoaling protection at the entrance channel as well as wave protection at the creek mouth and lower reaches of the creek. All the improvement plans tested, to this point, involved the use of sheet-pile (including cellular sheet-pile) structures. Considerable wave energy was observed reflecting off these structures, which could possibly stimulate erosion in the breakwater vicinity and affect navigation of small boats entering and leaving the harbor. Therefore, the sheet-pile structures for the most promising improvement plan tested were replaced with rubble-mound breakwaters. The rubble-mound breakwater plan reduced reflections in the immediate vicinity. It also provided slightly more wave protection to the creek mouth and lower reaches of the creek, and comparable shoaling protection at the entrance, when compared to the sheet-pile plan. The rubble-mound breakwater was more effective for the passage of flood flows, since some flow escaped through the voids of the structures. Tracer deposits for test waves from west-southwest are shown in Figure A-37 for this breakwater plan. Ice flow tests indicated no ice jamming tendencies at the entrance. A contract was awarded

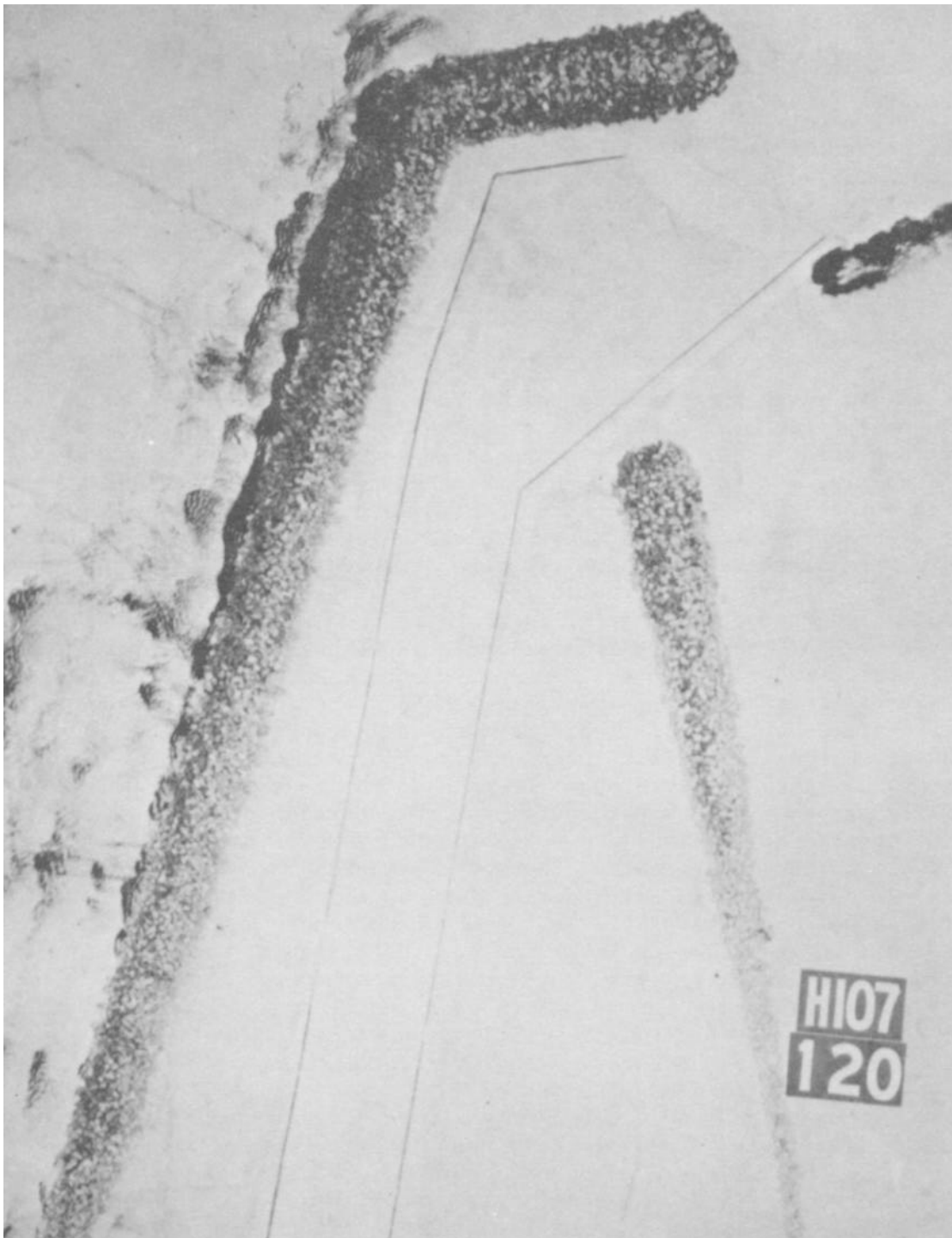


Figure A-37. Tracer deposits for the recommended improvement plan, Cattaraugus Creek, New York.

25 Sep 84

early in 1982 for for construction of improvements in the prototype at the mouth of Cattaraugus Creek, New York. Improvements constructed in the prototype (Figure A-38) were similar to those recommended by the hydraulic model investigation.



Figure A-38. Aerial photograph of Cattaraugus Creek mouth after improvements.

A-10. Entrance/Inlet Studies. Numerous small-craft harbors are located in inlet lagoons along the ocean coasts. Studies are frequently conducted to reduce navigational difficulties, shoaling, shoreline erosion, cross-currents, etc., at the entrance and to stabilize the inlet openings. Newburyport Harbor, Massachusetts, and Murrells Inlet, South Carolina, were selected as representative of this classification and are discussed below:

a. Newburyport Harbor, Massachusetts (Curren and Chatham 1979).

(1) The Prototype. Newburyport Harbor is located on the coast of Massachusetts, about 54 miles by water north of Boston and 20 miles southwest of Portsmouth, New Hampshire (Figure A-39). Newburyport Harbor was constructed during the period July 1881-October 1914. The city of Newburyport is the principal business center for several nearby towns and the summer resorts of Plum Island and Salisbury Beach, which are situated on the south and north sides, respectively, of the entrance to Newburyport Harbor.

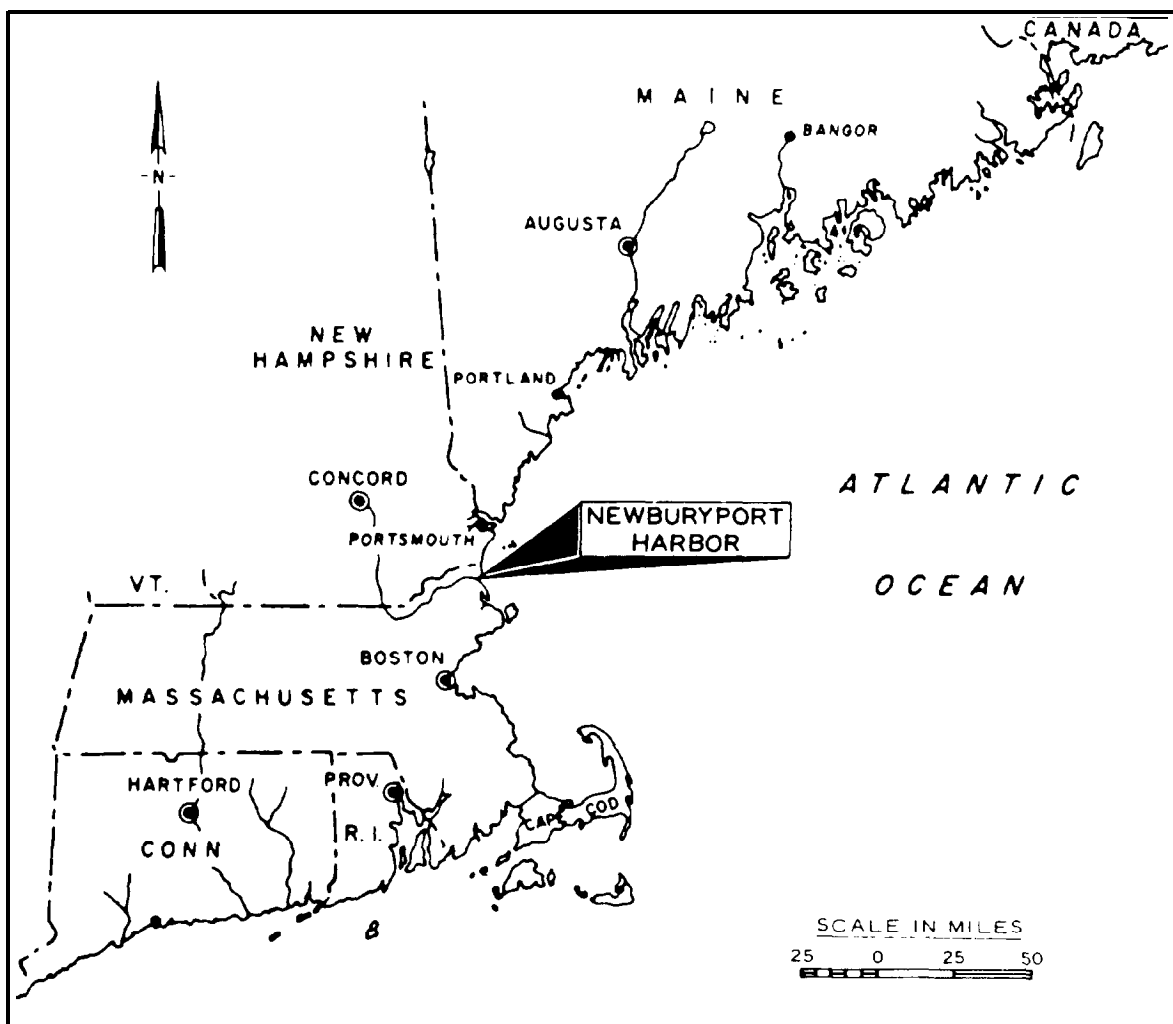


Figure A-39. Project location, Newburyport Harbor, Massachusetts.

(2) The Problem. Between 19 and 27 February 1969, three large storms entered the Merrimack Embayment and caused irreparable damage to the riverbank inside the south jetty. Waves overtopping the north jetty eroded approximately 260 feet of sand from the front of the U. S. Coast Guard Station located there; the resulting loss of sand totaled about 1,080,000 cubic yards. In an attempt to halt the erosion process, a revetment was installed in front of the Coast Guard Station. The effect of this revetment was a transfer of the problem upriver.

(3) The Model and Test Conditions. A physical model investigation was conducted to determine the mechanisms by which sand is being lost from the riverbank inside the south jetty, and to evaluate the effects of various improvement plans with respect to shoaling, riverbank erosion, wave conditions,

and construction costs. The Newburyport Harbor model (Figure A-40) was constructed to an undistorted linear scale of 1:75, model to prototype. Model test waves ranging from 7-13 seconds and 4-18 feet shown in Table A-7 were used during model operation. Still-water levels (swl) were selected to

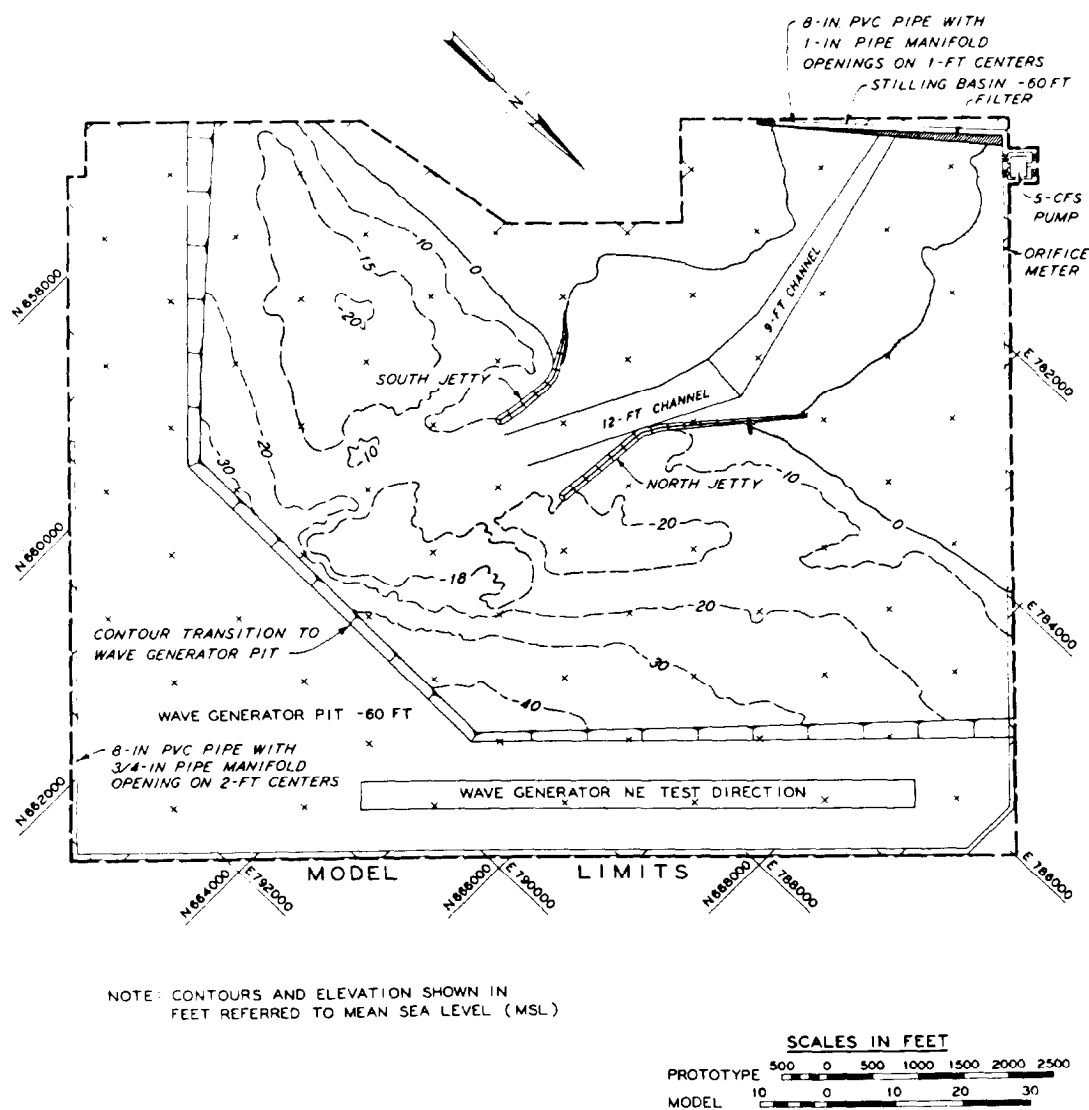


Figure A-40. Model layout, Newburyport Harbor.

TABLE A-7

Test Waves Used in the Newburyport Harbor Model (NOA)
(NOAA, 1976)

<u>Deepwater Wave Direction</u>	<u>Selected Shallow-Water Wave Test Direction (deg)</u>	<u>Selected Test Wave</u>	
		<u>Period (sec)</u>	<u>Height (ft)</u>
NE(39.5°)	51	7	5, 8, 11
		11	6, 9, 15
		15	11
E(89.5°)	90	7	4, 8, 12
		11	7, 11, 14, 18
SE(139.5°)	122	6	4, 8, 12
		9	4, 8, 12
		13	6

correspond with maximum steady state ebb and flood tidal velocities. From prototype data, maximum ebb current velocities occurred at a swl of 0.0 msl (mean sea level). Maximum flood velocities occurred at a swl of +2.9 feet. Also selected for testing was a slack water condition at a swl of +5.3 feet mhw (mean high water). A water circulating system was used in the model to reproduce these ebb and flood tidal flows and an automated data acquisition and control system (ADACS) was utilized to secure wave height data. A quantity of crushed coal tracer was used to determine qualitatively the movement of sediments. A general view of the model is shown in Figure A-41.

(4) Tests and Results.

(a) Existing Conditions. Prior to tests of various improvement plans, comprehensive tests were performed for existing conditions to determine wave and current conditions and tracer patterns. Test results indicated, for moderate to large incident waves, turbulent wave conditions in the entrance channel and strong longshore currents in the area between the south jetty and Plum Island Point, resulting in continued northeasterly movement of tracer material along the eroding portion of Plum Island (Figure A-42).

(b) Improvement Plans. Wave heights, current patterns and magnitudes, and tracer tests were conducted for 13 improvement plan variations. These variations consisted of changes in the length of the north jetty, changes in the crown elevation of the north jetty, and the installation of groins at two locations. Raising the elevation of the existing north jetty to +11.0 feet improved entrance wave conditions by preventing overtopping of the jetty by

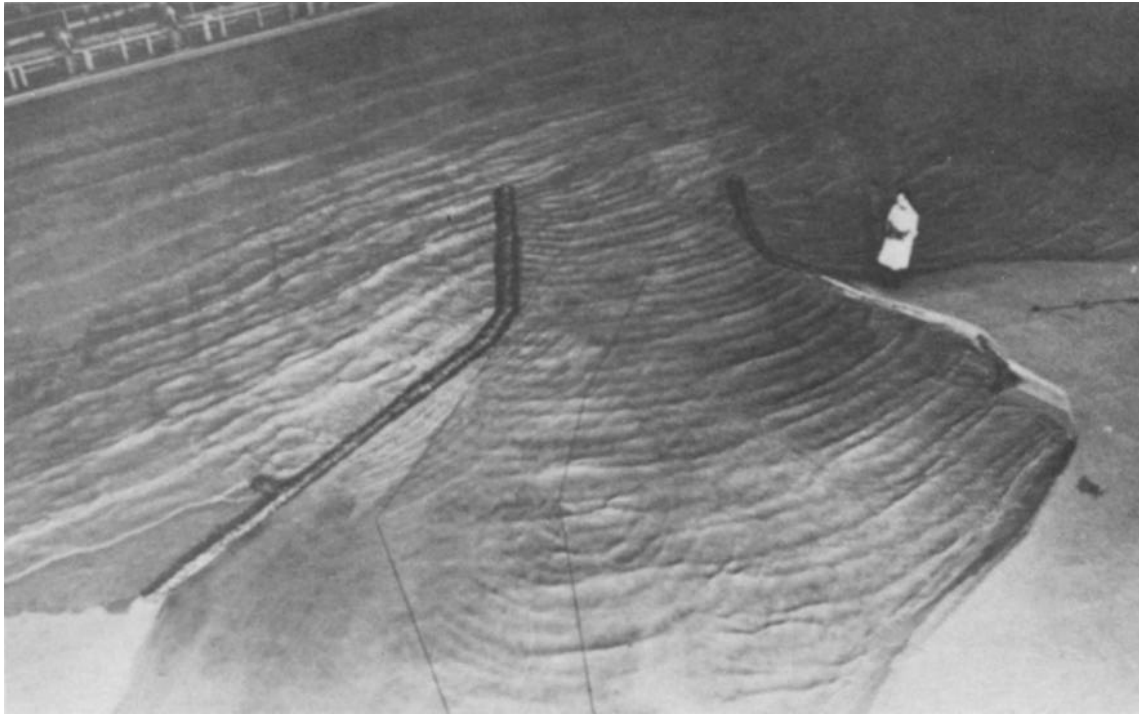


Figure A-41. General view of model, Newburyport Harbor.



Figure A-42. Typical tracer movement for existing conditions, Newburyport Harbor.

storm waves. This not only decreased the magnitude of the waves but also the turbulence created by overtopping waves interacting with waves traveling through the entrance. The installation of the groin from the area of Plum Island experiencing erosion, effectively prevented any further erosion from occurring for all wave and tidal flow conditions. In fact, for many cases, the groin actually accreted material. Of the plans tested, Plan 3A (Figure A-43) offers adequate erosion protection while improving entrance wave conditions and appears to be the optimum plan with regard to protection provided and cost.

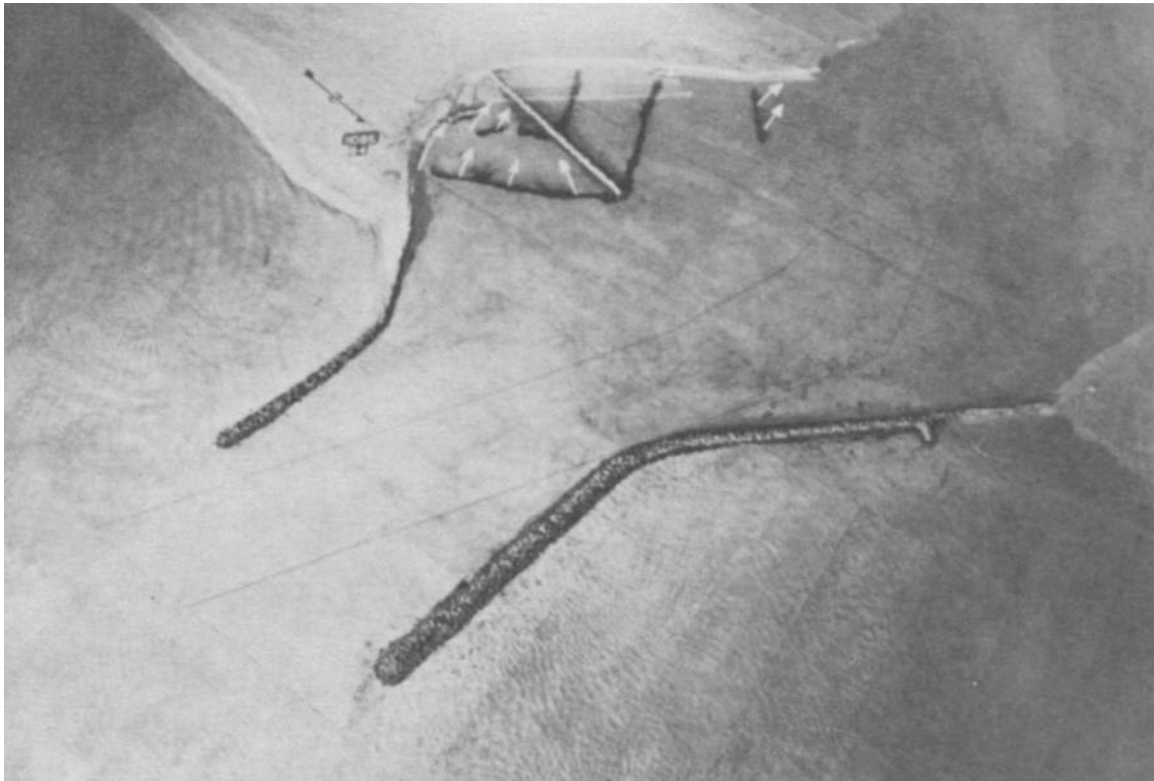


Figure A-43. Recommended improvement plan, Newburyport Harbor, Massachusetts.

b. Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978).

(1) The Prototype. Murrells Inlet was an unimproved inlet through the beachline of South Carolina about 19 miles northeast of the city of Georgetown, South Carolina, and 13 miles southwest of Myrtle Beach, South Carolina. The inlet provides access to a well-mixed tidal lagoon of ocean salinity that has no source of freshwater inflow other than local surface runoff. The inlet maintains its existence due to tidal current generated by the ocean tidal height variation (mean ocean tide range is 4.8 feet) which generates ebb and

flood currents that transport a tidal prism of 253 million cubic feet flowing through the inlet during a tidal cycle of 12.42 hours. In opposition to the tidal currents that tend to maintain an open inlet are littoral currents generated by waves carrying sand along the shoreline into the vicinity of the inlet, causing the formation of shallow regions of sand shoals. The inlet is used extensively by charter fishing craft, private boats, and commercial fishing vessels. Also the inlet and lagoon are environmentally important as a habitat and nursery for many varieties of marine life.

(2) The Problem. Unstabilized inlets, such as Murrells Inlet, can migrate along the coastline. Over about the last 100 years the inlet has varied in location by as much as 7000 feet. The pre-project conditions at the inlet produced a difficult and dangerous navigational environment as the main channel could vary in location and depth very quickly. Breaking waves on the shallow shoals, combined with the above conditions could produce very hazardous navigation as the inlet was unprotected and exposed to all Atlantic Coast waves. Waves normally range from 2 to 4 feet, but much larger waves are not unusual.

(3) Possible Solutions. Usually tidal inlet entrance improvements include the use of jetties, normally constructed of rock rubble, which attach to the shoreline and approximately parallel to the navigation channel seaward to the ocean contour of the depth of the design channel. There are usually a number of jetty alignments which may fit a given situation. The jetties main purpose is to prevent longshore sediments from shoaling the channel and offer protection from waves for incoming and departing vessels. More recently jetty design has taken the problem of littoral drift into consideration by providing weir sections in the jetties and sediment traps adjacent to the weir in which to capture the longshore drift, thus keeping the sediment out of the channel and also placing it in a location where it can be handled and available for future beach nourishment. The Murrells Inlet study provides such an example.

(4) The Model. A physical model was used to study and find the optimum alignment and spacing of the jetties, determine proper channel alignment and current patterns at the entrance, study effects on the tidal prism and bay tidal elevations and velocities, and determine wave heights in the entrance channel and deposition basin. A distorted scale model of 1:200 horizontal and 1:60 vertical scales was selected (Figure A-44). The entire lagoon was modeled to permit the study of the tidal elevations and currents and the tidal prism. A distorted scale model must be verified for its tidal currents and elevations, so prototype measurements of these parameters were required. Data were taken at locations seen in Figure A-44 and reproduced in the model by the adjustment of roughness elements that usually are required in distorted models.

(5) Testing. After tidal verification, numerous jetty plans were installed in the model for testing. The preliminary testing consisted of measuring wave heights at a variety of locations in the entrance channel and inner channels for various test waves at various stages of the tidal cycle, measuring

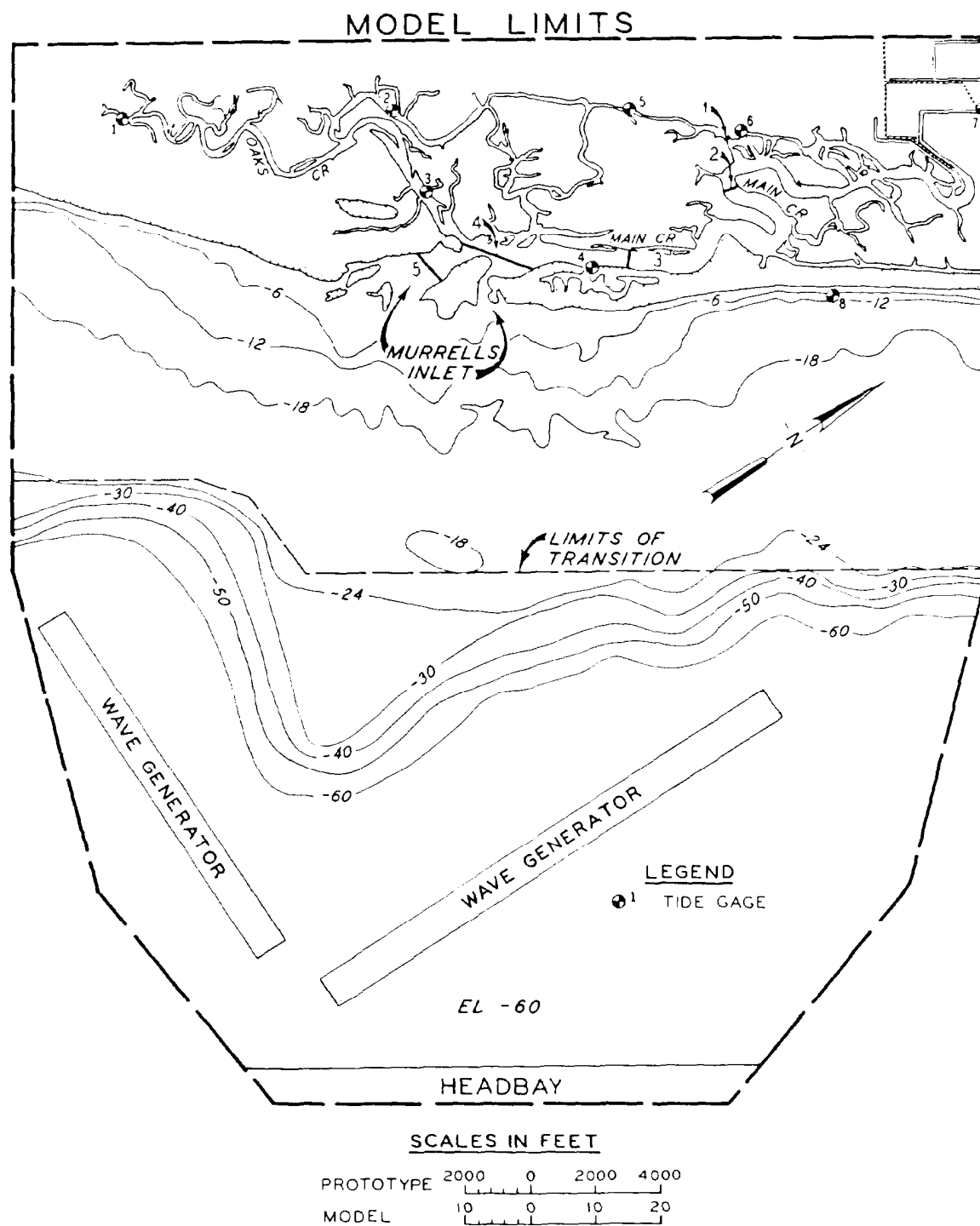


Figure A-44. Model layout, Murrells Inlet, South Carolina.

tidal elevations at the various verified locations for the entire tidal cycle, and taking surface current photographs at the entrance throughout the tidal cycle. Examination of these preliminary data permitted reducing the number of plans which would be submitted to more testing that included detailed current measurement and wave height measurements. Further refinements could then be made in the design. For example Plan 1B (Figure A-45) was selected for further testing and gradually evolved into Plan 1H (Figure A-46) as changes were made in the widths and depths of the inner auxillary channels (which connect the main navigation channel to the interior bay channels) to improve flow patterns and flow admittance; the jetty spacing was reduced from 900 feet to 600 feet to provide adequate scouring currents in the channel but still maintain a similar tidal prism to that of the pre-jetty conditions; the access channel to the deposition basin was relocated; and a training dike was added to prevent ebb currents from entering the region of the deposition basin.

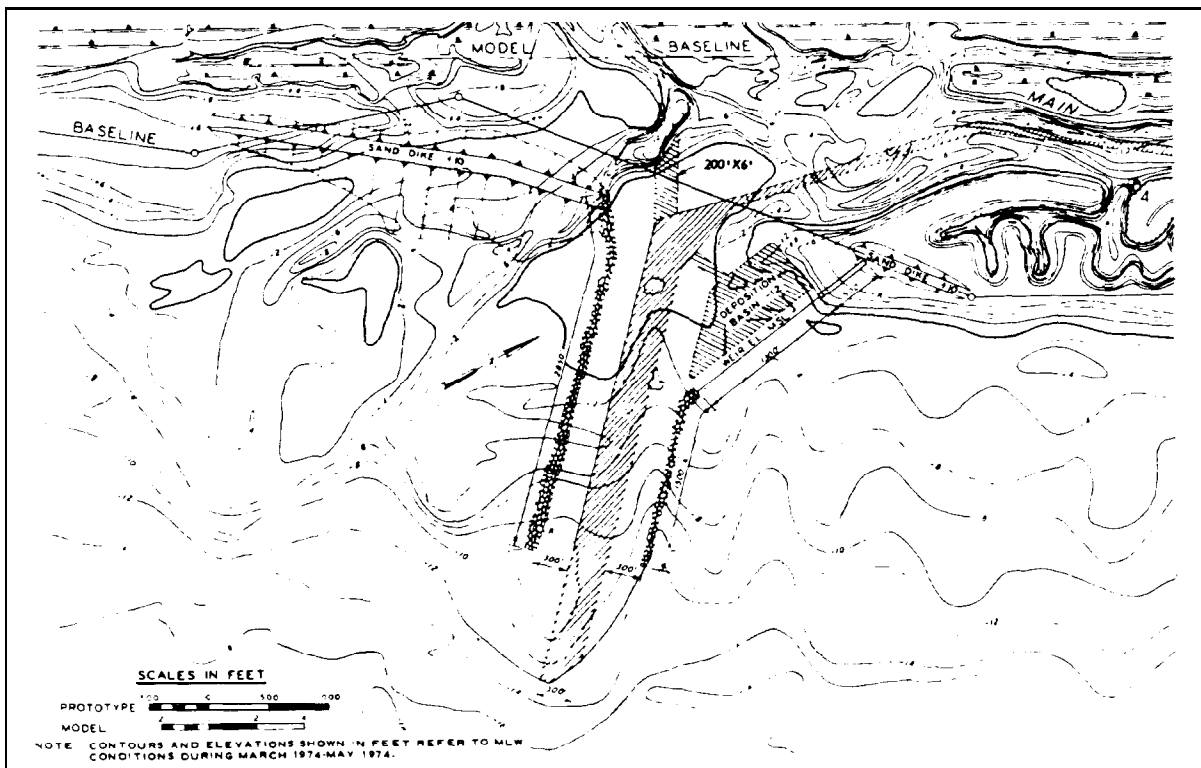


Figure A-45. Typical plan of improvement for Murrells Inlet.

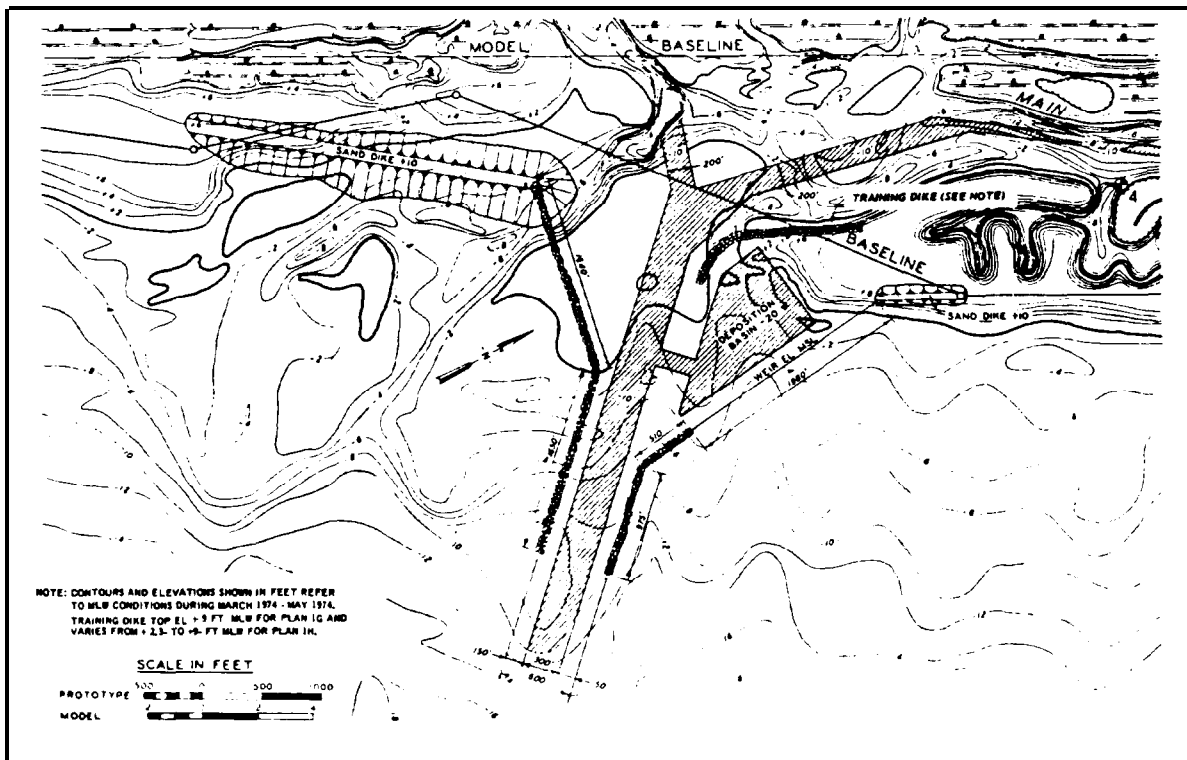


Figure A-46. Optimum improvement plan, Murrells Inlet, South Carolina.

Figure A-47 shows the project which was completed in January 1981. The only element of the plan not constructed was the training dike which may be added at a later data if required. As can be seen, the deposition basin is filling and to date the navigation channel has naturally maintained depths greater than the project depth.



Figure A-47. View of Murrells Inlet project, as constructed in 1981.